HYDRAULIC DESIGN CRITERIA

SHEET 000-1

PHYSICAL CONSTANTS

ACCELERATION OF GRAVITY

EFFECTS OF LATITUDE AND ALTITUDE

1. The value of acceleration of gravity commonly quoted in hydraulics textbooks is 32.2 ft/sec^2. Accordingly, the value of 2g in conversions between velocity and velocity head would be 64.4 ft/sec^2. Some engineers prefer to use 64.3 ft/sec^2 as being more representative of the acceleration of gravity for the United States.

2. Hydraulic Design Chart 000-1 was prepared to afford the engineer a convenient illustration of the nature of the variation of the acceleration of gravity with latitude and altitude. The theoretical values of acceleration of gravity at sea level are based on the international gravity formula converted to English units(2)

$$g_o = 32.08822\ (1 + 0.0052884 \sin^2 \phi - 0.0000059 \sin^2 2\phi)$$

where

g_o = acceleration of gravity at sea level in ft/sec^2
ϕ = latitude in degrees.

Tabular values are given in reference (2). The correction for elevation above sea level is contained in the equation:

$$g_H = g_o - 0.000003086\ H$$

where

g_H = acceleration of gravity at a given elevation in ft/sec^2
H = elevation above sea level in ft.

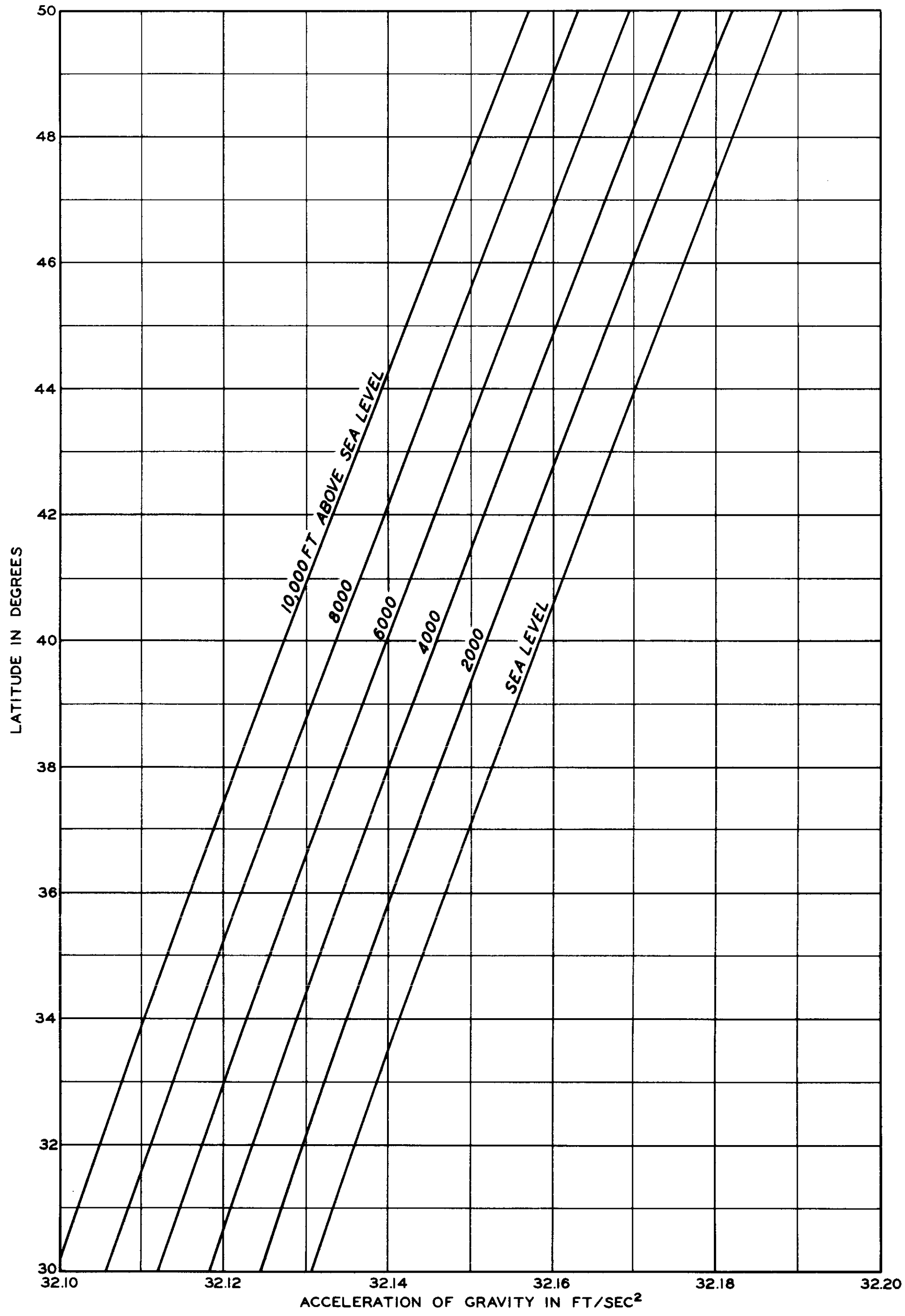
3. Chart 000-1 presents the variation of the acceleration of gravity with altitude for north latitudes from 30-50 degrees. The value of g for sea level at the equator is 32.088 ft/sec^2 and at Fairbanks, Alaska, is 32.227 ft/sec^2.

4. The values of the acceleration of gravity as measured by a pendulum are available from the Coast and Geodetic Survey.(1) The deviation of the measured value from the theoretical value, corrected for altitude, is called the free air anomaly. A plus or minus anomaly of 0.0016 ft/sec^2 may be considered large, except in high mountains or deep gorges.

5. References.

(1) Duerksen, J. A., Pendulum Gravity Data in the United States. U. S. Coast and Geodetic Survey Special Publication No. 244, 1949.

(2) Swick, C. H., Pendulum Gravity Measurements and Isostatic Reductions. U. S. Coast and Geodetic Survey Special Publication No. 232, 1942.



NOTE: CHART PREPARED FROM INFORMATION PUBLISHED IN USC & GS SPECIAL PUBLICATION NO. 232, "PENDULUM GRAVITY MEASUREMENTS AND ISOSTATIC REDUCTIONS," BY C. H. SWICK, 1942.

PHYSICAL CONSTANTS

ACCELERATION OF GRAVITY
EFFECTS OF LATITUDE AND ALTITUDE

HYDRAULIC DESIGN CHART 000-1

HYDRAULIC DESIGN CRITERIA

SHEET 000-2

PHYSICAL CONSTANTS

BAROMETRIC DATA

ALTITUDE VS PRESSURE

1. Cavitation. The equation for incipient cavitation index takes into account the vapor pressure of water:

$$K_i = \frac{h_o - h_v}{V_o^2/2g}$$

where h_o is the absolute pressure in ft of water, h_v is vapor pressure of water in ft, and V_o is velocity of the water in ft per sec.

2. Vapor Pressure. The vapor pressure of water has been found to vary with the temperature as follows(1,2,3):

Temp, F	h_v ft of Water Absolute
32	0.20
50	0.41
70	0.84

3. Barometric Pressure. The value of the numerator in the above equation is also dependent upon h_o which is the barometric pressure less the negative pressure measured from atmospheric pressure. The incipient cavitation index is thus dependent upon the barometric pressure. For similar boundary geometry and similar flow conditions, the chances of cavitation occurring are somewhat greater at higher altitudes than at lower altitudes. The effect of altitude on cavitation possibilities is more marked than the effect of temperature.

4. Chart 000-2. The variation of barometric pressure with altitude is given on Chart 000-2. This chart was plotted using values given by King (reference 2, page 18), and agrees very closely with the values presented by the Smithsonian Institute (reference 1, page 559).

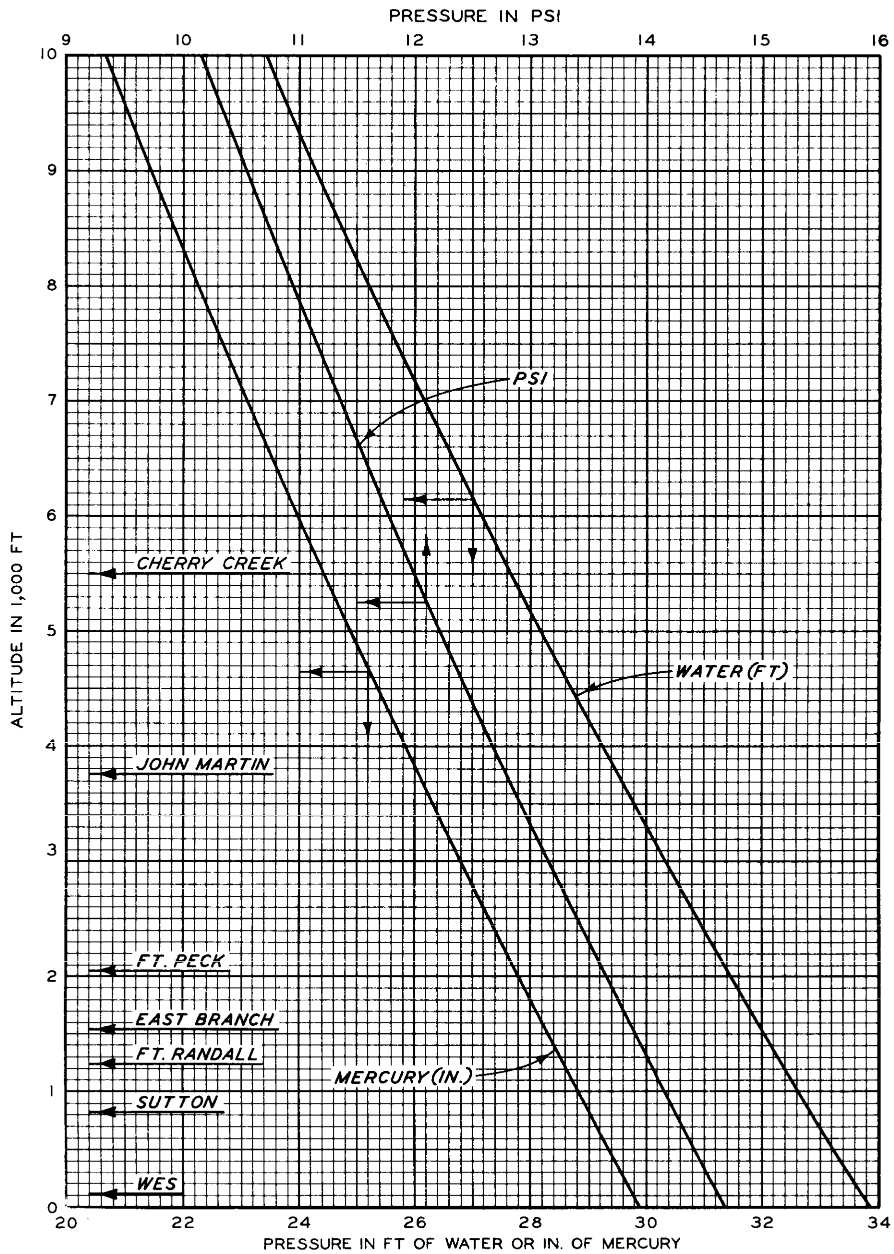
5. Other Applications. Barometric pressure is also of interest in the vertical limit of pump suction lines and turbine draft tubes.

6. References.

(1) Fowle, F. E., Smithsonian Physical Tables. Vol 88, Smithsonian Institute, Washington, D. C., 1934, p 232, p 559.

(2) King, H. W., Handbook of Hydraulics. 3d ed., McGraw-Hill Book Co., Inc., New York, N. Y., 1939, table 14, p 18.

(3) National Research Council, International Critical Tables. Vol III, McGraw-Hill Book Co., Inc., New York, N. Y., 1928, p 211.



NOTE: PRESSURES ARE FOR AIR TEMPERATURE OF 50 F.

PHYSICAL CONSTANTS
BAROMETRIC DATA
ALTITUDE VS. PRESSURE

HYDRAULIC DESIGN CHART 000-2

REVISED 5-59 WES 8-58

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

HYDRAULIC DESIGN CRITERIA

SHEETS 001-1 to 001-5

FLUID PROPERTIES

EFFECT OF TEMPERATURE

1. Data on the fluid properties of water are required for the solution of many hydraulic problems. Hydraulic Design Charts 001-1 through 001-5 present information on those properties most commonly used in the design of hydraulic structures, and are included to afford convenient references for the design engineer.

2. Charts 001-1, 001-2, and 001-3 show the effect of temperature on kinematic viscosity, vapor pressure, and surface tension of water. The freshwater data on the charts, in the order numbered, were prepared from data published in the International Critical Tables (references 4 and 5, 3, and 4, respectively). The saltwater data on Chart 001-1 is from reference 6.

3. Chart 001-4 presents bulk modulus of water curves at atmospheric pressure for temperatures of 32° to 100° F. The Randall and Tryer curves were plotted from data published by Dorsey (reference 1). The National Bureau of Standards curve was computed from Greenspan and Tschiegg data (reference 2) on the speed of sound in water. The equation used in the computation was

$$V = \sqrt{\frac{E}{\rho}}$$

where

V = speed of sound in water in ft per sec
E = bulk modulus in psi
ρ = density of fluid in slugs per cu ft

A change in pressure up to 10 atmospheres appears to have negligible effect on the value of the bulk modulus.

4. A curve for the Greenspan and Tschiegg data on the effect of temperature on the speed of sound in water is shown on Chart 001-5.

5. References.

(1) Dorsey, N. Ernest, Properties of Ordinary Water-Substance, Reinhold Publishing Corp., New York, N. Y., 1940, Table 105, p 243.

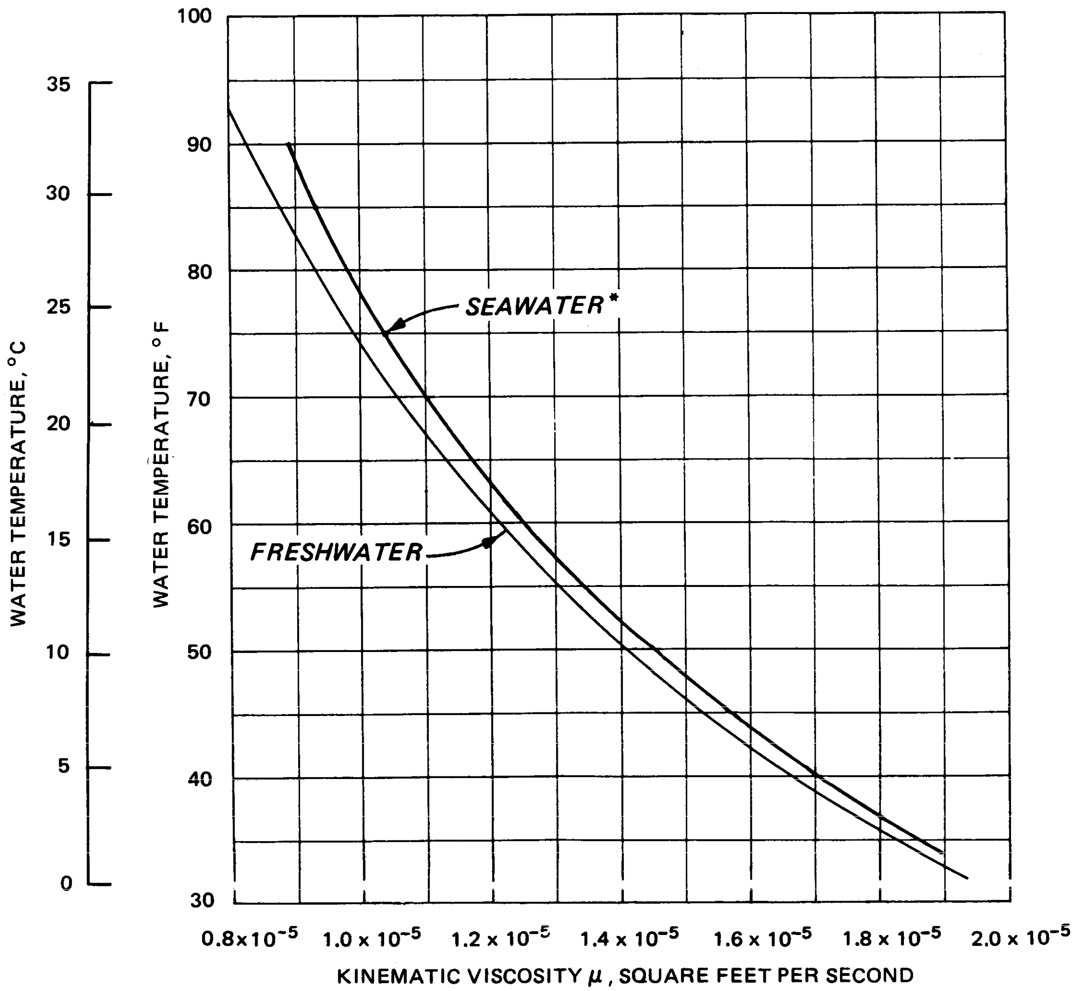
(2) Greenspan, M., and Tschiegg, C. E., "Speed of sound in water by a direct method." Research Paper 2769, Journal of Research of the National Bureau of Standards, vol 59, No. 4 (October 1957).

(3) International Critical Tables, vol III, First Edition, McGraw-Hill Book Co., Inc., New York and London, 1928, p 211 (vapor pressure).

(4) International Critical Tables, vol IV, First Edition, McGraw-Hill Book Co., Inc., New York and London, 1928, p 25 (density) and p 447 (surface tension).

(5) __________, vol V, First Edition, McGraw-Hill Book Co., Inc., New York and London, 1929, p 10 (dynamic viscosity).

(6) Saunders, H. E., Hydrodynamics in Ship Design, Society of Naval Architects and Marine Engineers, 1964.

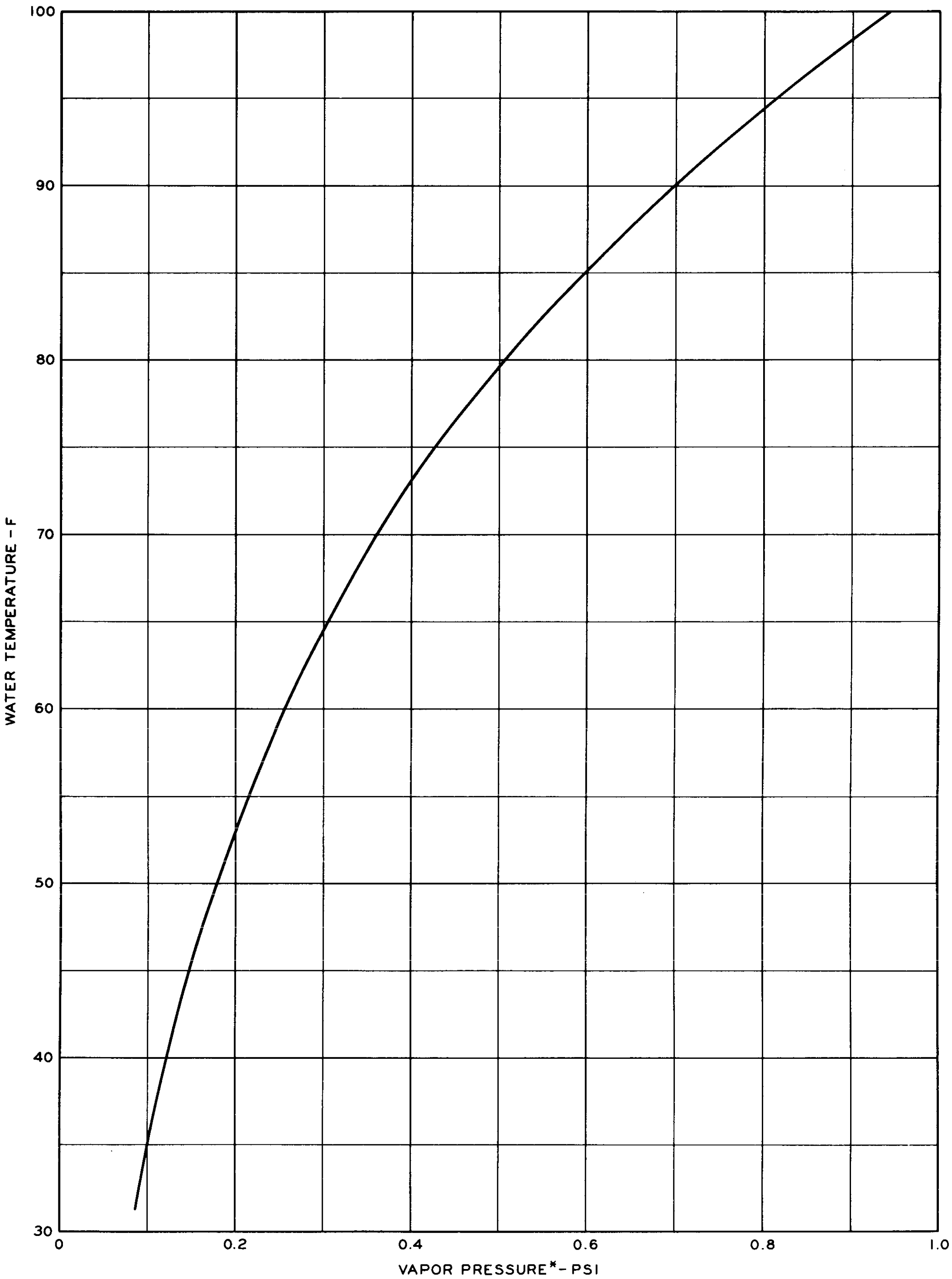


REFERENCES: INTERNATIONAL CRITICAL TABLES, VOL IV, PAGE 25 (REFERENCE 4); VOL V, PAGE 10, FIRST EDITION (REFERENCE 5); AND SEAWATER DATA FROM HYDRODYNAMICS IN SHIP DESIGN, BY H.E. SAUNDERS (REFERENCE 6).

* 35 PARTS PER THOUSAND SALINITY (BY WEIGHT)

FLUID PROPERTIES

KINEMATIC VISCOSITY OF WATER
EFFECT OF TEMPERATURE
HYDRAULIC DESIGN CHART 001-1



REFERENCE: <u>INTERNATIONAL CRITICAL TABLES</u>, VOL. III, PAGE 211, FIRST EDITION

*FRESH WATER

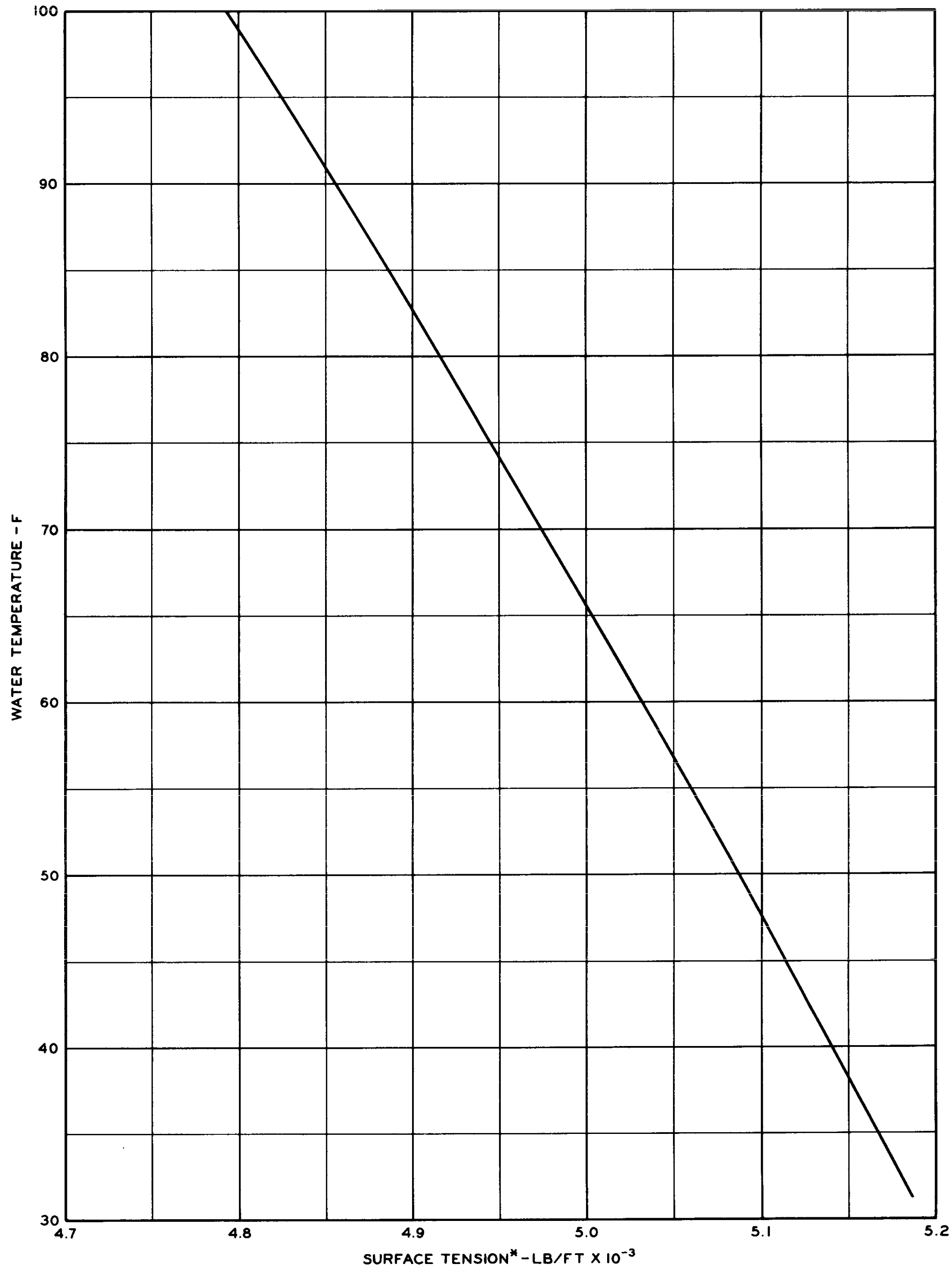
FLUID PROPERTIES

VAPOR PRESSURE OF WATER EFFECT OF TEMPERATURE

HYDRAULIC DESIGN CHART 001-2

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 8-60



REFERENCE: INTERNATIONAL CRITICAL TABLES, VOL. IV, PAGE 447, TABLE A-B, FIRST EDITION

*FRESH WATER

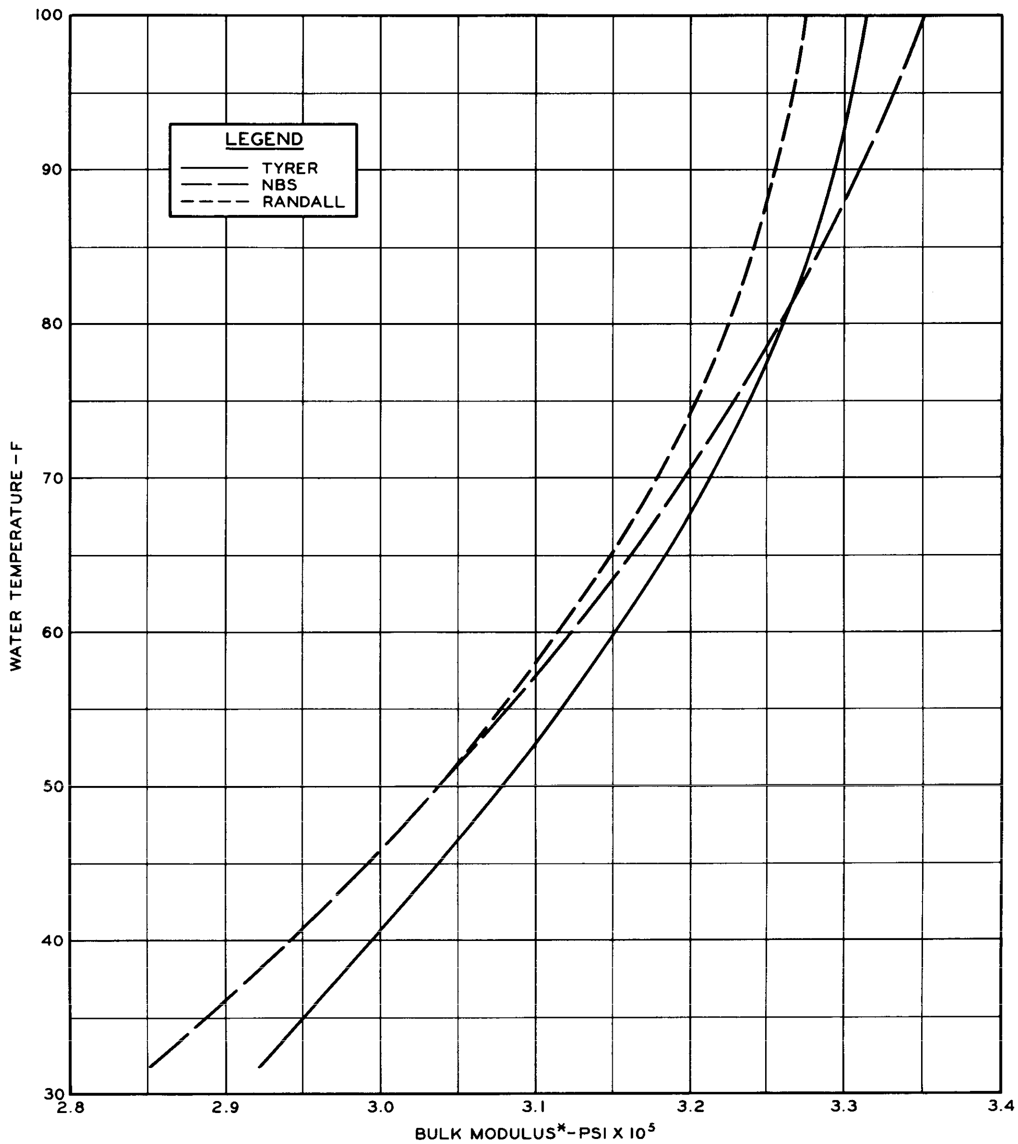
FLUID PROPERTIES

SURFACE TENSION OF WATER EFFECT OF TEMPERATURE

HYDRAULIC DESIGN CHART 001-3

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 8-60



NOTE: CURVES SHOWN ARE FOR ATMOSPHERIC PRESSURE.

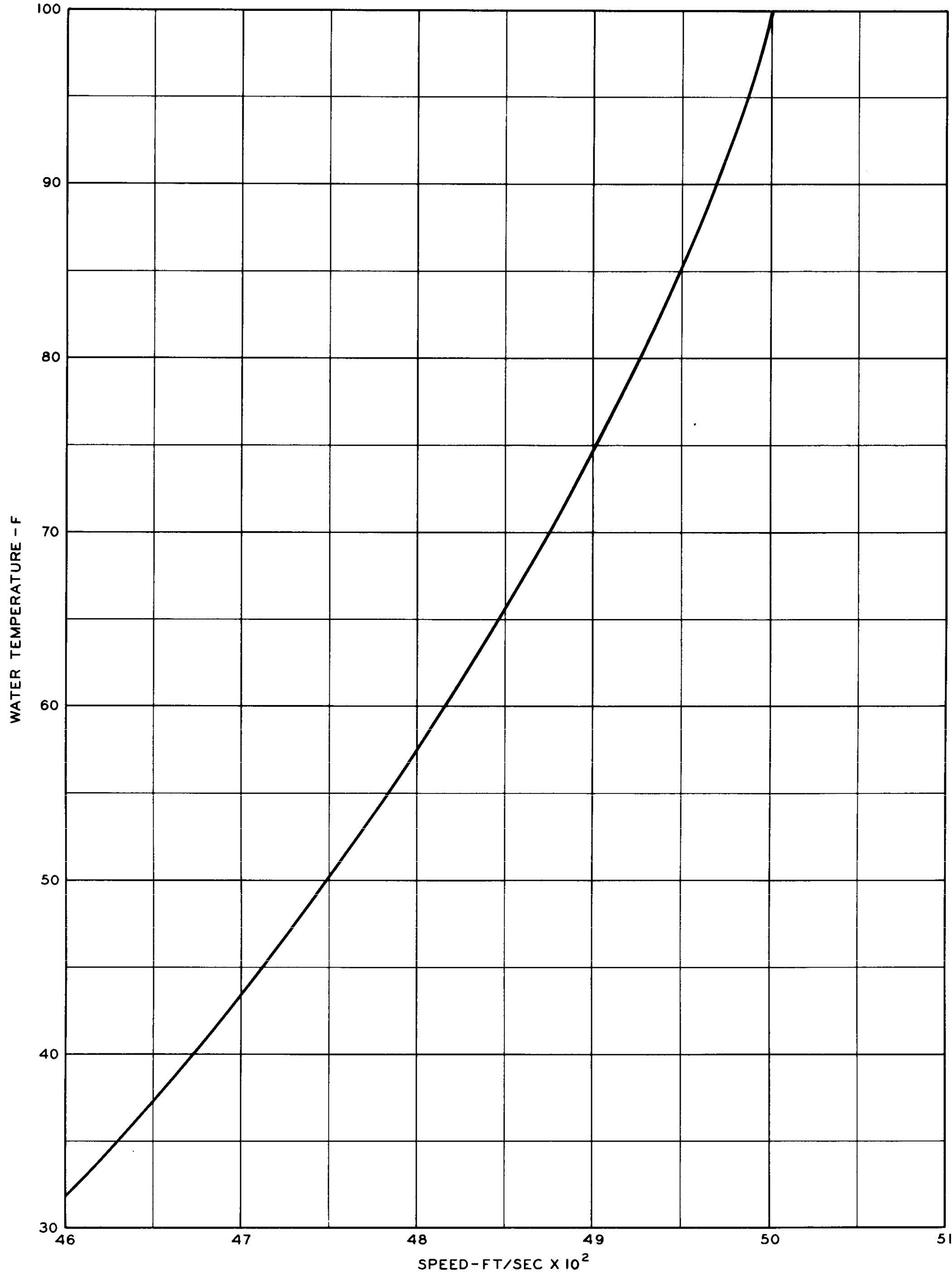
*FRESH WATER

FLUID PROPERTIES

BULK MODULUS OF WATER EFFECT OF TEMPERATURE

HYDRAULIC DESIGN CHART 001-4

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI



NOTE: CURVE PLOTTED FROM DATA REPORTED BY M. GREENSPAN AND C. E. TSCHIEGG; JOUR. OF RES., NBS, VOL. 59, NO. 4, 1957, ON DISTILLED WATER.

FLUID PROPERTIES

SPEED OF SOUND IN WATER
EFFECT OF TEMPERATURE

HYDRAULIC DESIGN CHART 001-5

HYDRAULIC DESIGN CRITERIA

SHEET 010-1

OPEN CHANNEL FLOW

SURFACE CURVE CLASSIFICATIONS

1. Bakhmeteff's treatise on open channel flow[1] illustrates and defines classifications of surface curves of nonuniform flow. Hydraulic Design Chart 010-1 presents definition sketches of six water-surface curves encountered in many design problems. Although this schematic representation of classification of surface profiles has been presented in numerous textbooks, it is included here for ready reference. In addition, the chart presents examples of each type of surface curve chosen from problems that commonly occur in the work of the Corps of Engineers.

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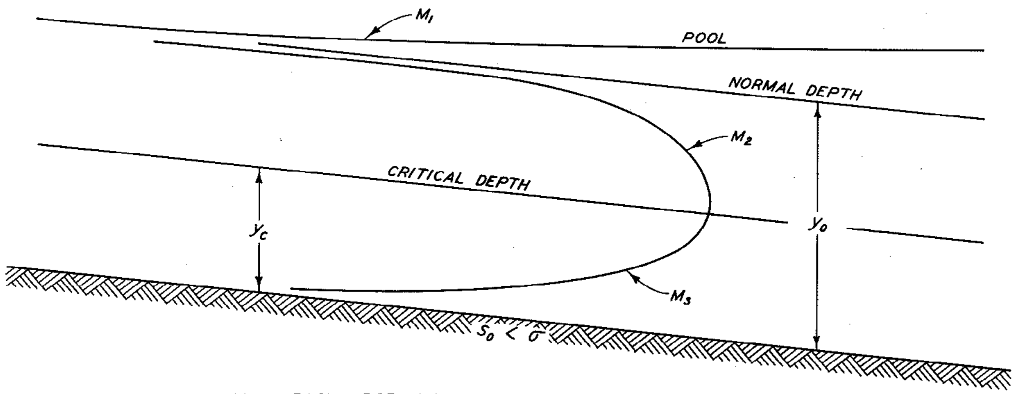
[1] B. A. Bakhmeteff, Hydraulics of Open Channel Flow, New York, N. Y., McGraw-Hill Book Company (1932), chapter VII.



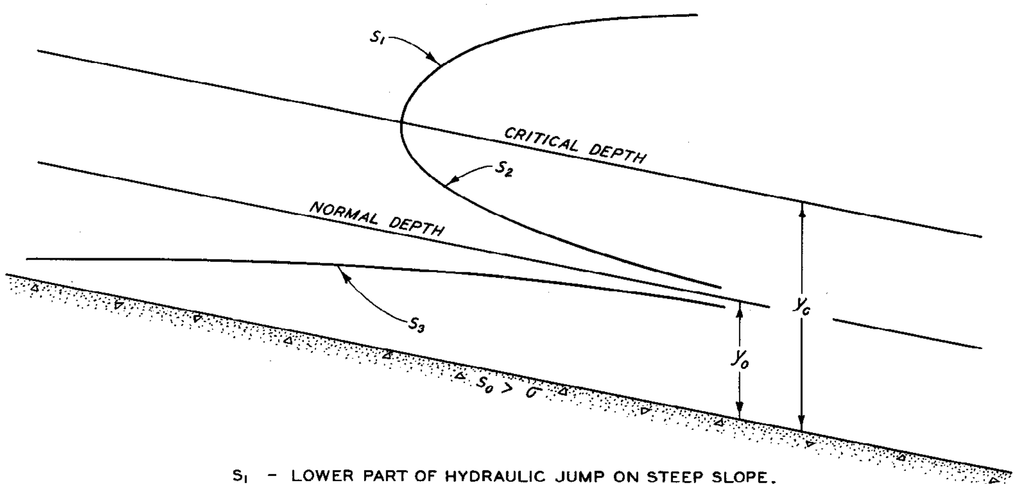
M_1 – BACKWATER FROM RESERVOIR – UNIFORM CHANNEL.
$y > y_0$, $\eta > 1$.

M_2 – DROP-DOWN TO SPILLWAY WITH SHALLOW APPROACH.
$y_0 > y > y_c$, $\eta < 1$.

M_3 – FLOW UNDER GATE ON MILD SLOPE.
$y < y_c$, $\eta < 1$.

MILD SLOPE

$y_0 > y_c$



S_1 – LOWER PART OF HYDRAULIC JUMP ON STEEP SLOPE.
$y > y_c$, $\eta > 1$.

S_2 – CHUTE FLOW FROM LOW OGEE CREST.
$y_c > y > y_0$, $\eta > 1$.

S_3 – CHUTE FLOW FROM HIGH OGEE CREST. FLOW UNDER GATE ON STEEP SLOPE.
$y < y_0$, $\eta < 1$.

STEEP SLOPE

$y_0 < y_c$

$$\eta = \frac{y}{y_0}$$

OPEN CHANNEL FLOW CLASSIFICATIONS
UNIFORM SLOPES

HYDRAULIC DESIGN CHART 010-1

WES 3-56

HYDRAULIC DESIGN CRITERIA

SHEETS 010-2 TO 010-5/3

OPEN CHANNEL FLOW

BACKWATER COMPUTATIONS

1. Hydraulic Design Charts 010-2 to 010-5/3 are aids for reducing the computation effort in the design of uniform channels having nonuniform flow. It is expected that the charts will be of value in preliminary design work where various channel sizes, roughness values, and slopes are to be investigated. Other features of the charts will permit accurate determination of water-surface profiles.

2. Basic Principle. The theoretical water-surface profile of nonuniform flow in a uniform channel can be determined only by integrating the varied-flow equation throughout the reach under study. Such integration can be accomplished by the laborious step method or by various analytical methods such as that of Bakhmeteff(1)* and others. However, all methods are tedious and, in many cases, involve successive approximations.

3. Escoffier(3) has developed a graphical method based on the Bakhmeteff varied-flow function. This method greatly simplifies varied-flow solutions and eliminates successive approximations. The desired terminal water-surface elevation of a reach can be determined with or without intermediate water-surface points. The method is adaptable to all uniform-channel flow problems except those with horizontal or adverse bottom slopes. The more elaborate method of Keifer and Chu(4) is indicated for problems with circular section when accuracy is required. The method may be used for natural water courses if the cross section and slope are assumed uniform and hydraulic shape exponents are determined. Precise determination of the hydraulic exponent is not necessary to assure appropriate accuracy in backwater computations for natural channels. Usually an average value within the indicated range of depths will suffice.

4. The equation developed by Bakhmeteff to compute the water-surface profile is:

$$L = \frac{y_o}{S_o} \left\{ (\eta_2 - \eta_1) - (1 - \beta) \left[B(\eta_2) - B(\eta_1) \right] \right\}$$

---

* Raised numbers in parentheses refer to list of references at end of text.

where

L, y_o, and S_o = length, normal depth, and bottom slope, respectively

η = dimensionless stage variable (y/y_o)

β = dimensionless quantity $(y_c/y_o)^N$

$B(\eta)$ = varied flow function (Chart 010-3)

y_c = critical depth

N = hydraulic exponent (Chart 010-4).

If the equation is divided by y_o/S_o then

$$L \frac{S_o}{y_o} = (\eta_2 - \eta_1) - (1 - \beta) \left[B(\eta_2) - B(\eta_1)\right] .$$

The term $\eta_2 - \eta_1$ is the vertical intercept of the varied-flow function plotted on Chart 010-3. The factor of the second term, $B(\eta_2) - B(\eta_1)$, is the horizontal intercept and $1 - \beta$ is a slope factor which converts the horizontal intercept into a component of the vertical. If the value of the equation computed from $L\, S_o/y_o$ is plotted vertically from a known η_1 on Chart 010-3 and the required slope line projected from the limit of this line to the curve, a value of η_2 is obtained from which the unknown depth can be computed, $y_2 = \eta_2\, y_o$.

5. Application. Chart 010-2 defines the equations required in the Escoffier graphical method and outlines the required steps in the solution. It is necessary to plot the slope line, $1 - \beta$, in accordance with the horizontal and vertical scales of the chart.

6. Chart 010-4 presents graphical plots of hydraulic exponents for different channel shapes for use in conjunction with Chart 010-3. The coordinates are in dimensionless terms and are therefore applicable to channels of various sizes. The equations for trapezoidal and circular shaped channels and the method of plotting were developed by N. L. Barbarossa(2). The general equation for hydraulic exponents applicable to all channel shapes was developed by Bakhmeteff.

7. Large-scale plots of the varied-flow function can be made where greater accuracy of results is required. Published tables(1) of the varied-flow function are given on Charts 010-5 to 010-5/3 for convenient reference. A hydraulic exponent of 3.3 is suggested for wide channels with two-dimensional flow. Tabulated values of the function for N = 3.3 were computed by the Waterways Experiment Station.

8. List of References.

(1) Bakhmeteff, B. A., Hydraulics of Open Channel Flow. McGraw-Hill Book Company, New York, N. Y., 1932.

(2) Barbarossa, N. L., Missouri River Division, CE, Omaha, Nebr., unpublished data.

(3) Escoffier, F. F., A Simplified Graphical Method for Determining Backwater Profiles. Mobile District, CE, Mobile, Ala., unpublished paper, 1955.

(4) Keifer, C. J., and Chu, H. H., "Backwater functions by numerical integration." Transactions, ASCE, vol 120 (1955), pp 429-448.

EQUATIONS AND DEFINITIONS

$\eta = \dfrac{y}{y_o}$, Where y = depth and y_o = normal depth.

$\beta = \left(\dfrac{y_c}{y_o}\right)^N$, Where y_c = critical depth and N = hydraulic exponent.

$I = \dfrac{LS_o}{y_o}$, Where L = length of reach and S_o = bottom slope.

$m = 1 - \beta$, Where m = slope of construction line, $\left(\dfrac{\Delta \eta}{\Delta B(\eta)}\right)$.

GENERAL APPLICATION

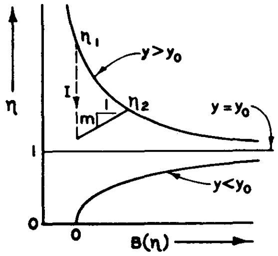
1. Given channel shapes, S_o, Q, y_1 and Manning's "n." Required to find y_2 at a distance L from y_1.

2. The following charts apply:

Channel Type	Design Criteria Chart y_o	y_c
Wide Channels	610-8	610-8
Rectangular Channels	610-9 and 9/1 *	610-8
Trapezoidal Channels	610-2 to 4/1 *	610-5 to 7
Circular Channels	224-8 *	224-9

* Also requires use of Charts 610-1 and 1/1.

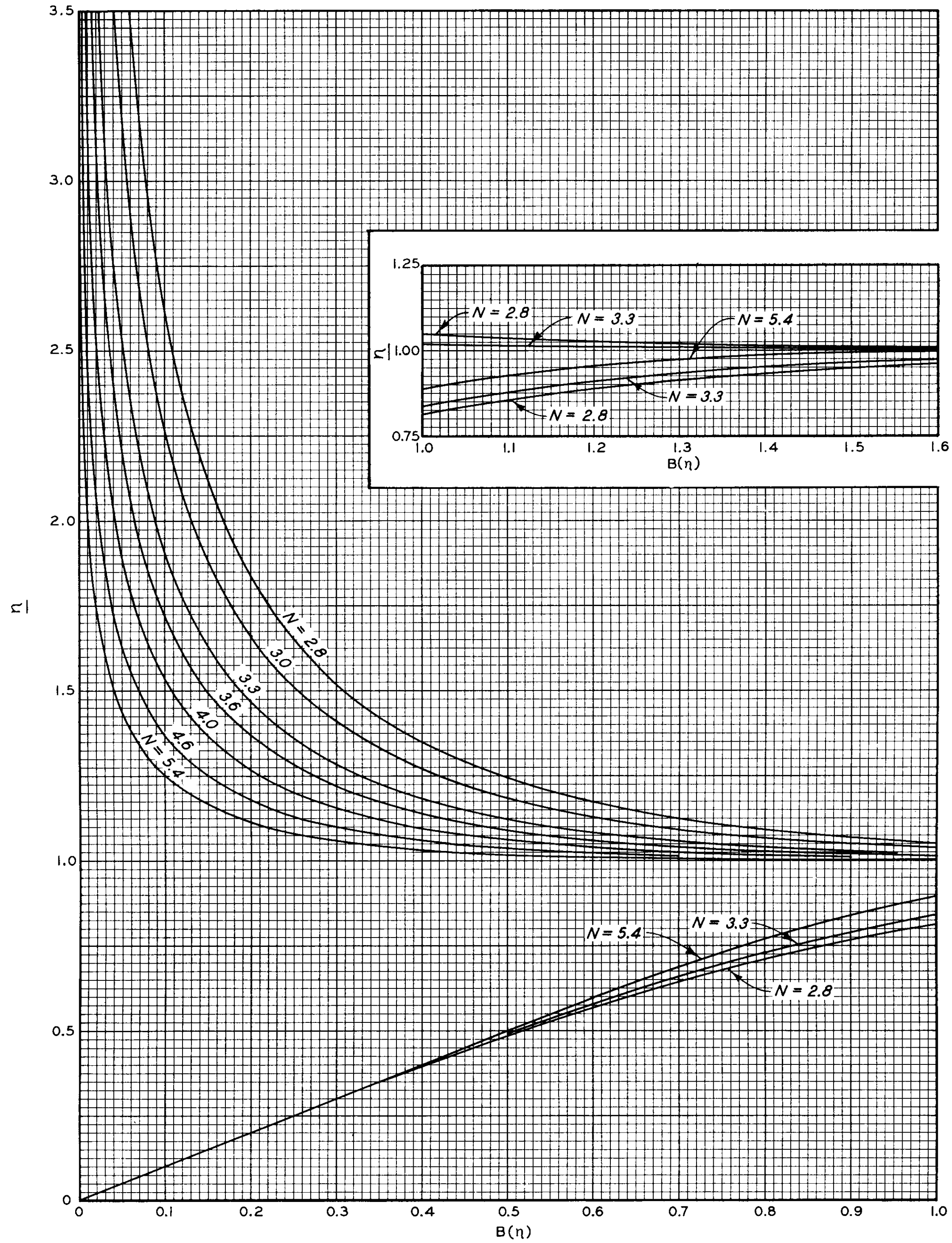
3. Select hydraulic exponent N from Chart 010-4.
4. Compute η_1, β, I and m from above equations.
5. Establish η_1 on curve of Chart 010-3 for proper value of exponent N.
6. Construct I in units of η vertically from η_1, upward for negative m and downward for positive m.
7. Draw slope line m through extremity of I and find η_2 where slope line intersects $B(\eta)$ curve.
8. Compute $y_2 = y_o \eta_2$.



OPEN CHANNEL FLOW

DEFINITION AND APPLICATION

HYDRAULIC DESIGN CHART 010-2



BASIC EQUATION

$$B(\eta) = -\int_0^{\eta} \frac{d\eta}{\eta^N - 1}$$

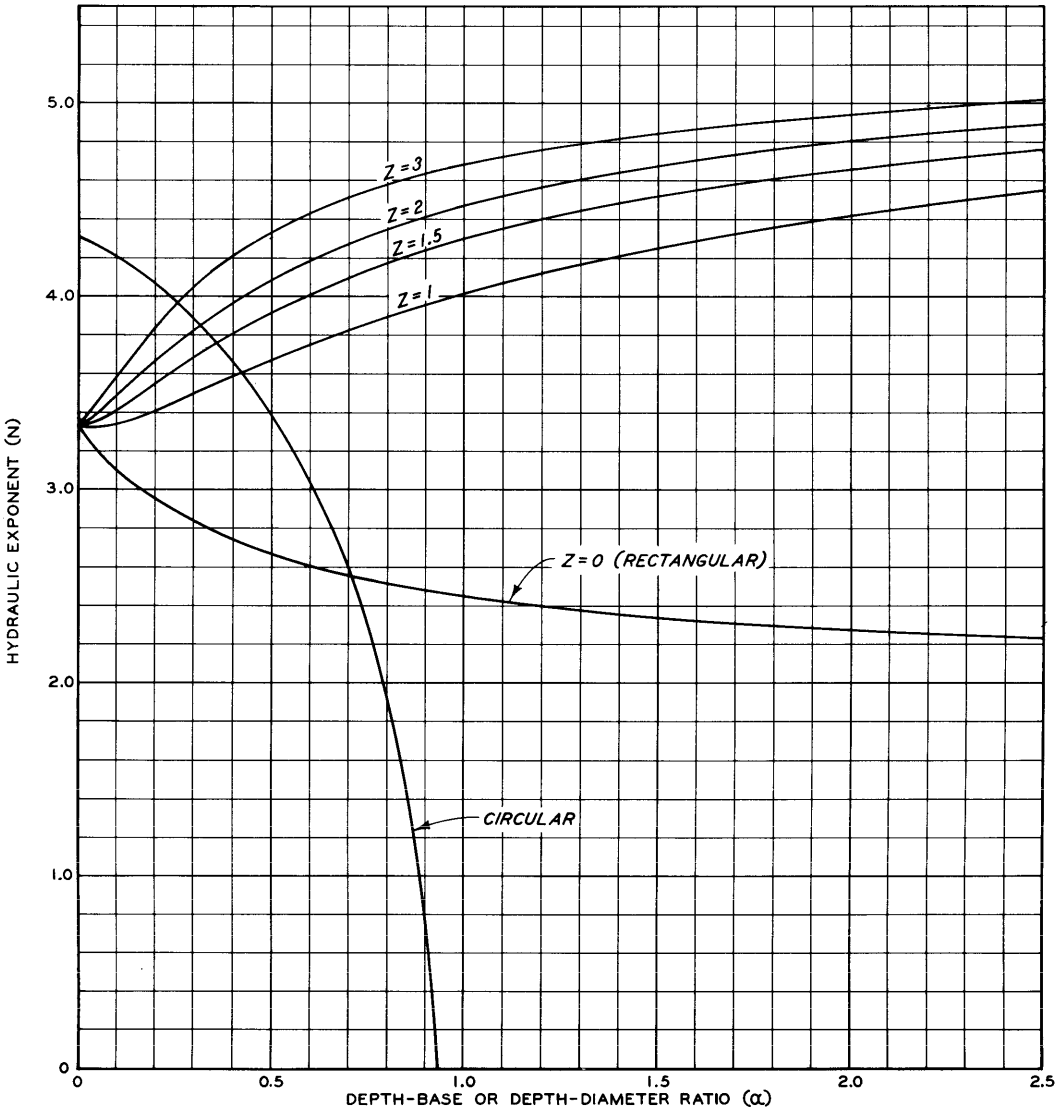
WHERE: $\eta = \frac{y}{y_0}$

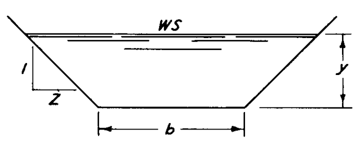
N = HYDRAULIC EXPONENT (3.3 FOR WIDE RECTANGULAR CHANNELS)

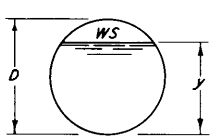
OPEN CHANNEL FLOW
η VS $B(\eta)$

HYDRAULIC DESIGN CHART 010-3

WES 3-56







GENERAL EQUATION

$$N = 2\frac{\mathrm{LOG}\,(K_2/K_1)}{\mathrm{LOG}\,(y_2/y_1)} = f(\alpha)$$

TRAPEZOIDAL SECTION

$$N = \frac{10}{3}\left(\frac{1+2Z\alpha}{1+Z\alpha}\right) - \frac{8}{3}\left(\frac{\alpha\sqrt{Z^2+1}}{1+2\alpha\sqrt{Z^2+1}}\right)$$

CIRCULAR SECTION

$$N = \frac{8}{3}\frac{1}{\sqrt{\alpha-\alpha^2}}\left[\frac{20\alpha^2(1-\alpha)}{\pi+4(2\alpha-1)\sqrt{\alpha-\alpha^2}+2\,\mathrm{SIN}^{-1}(2\alpha-1)} - \frac{\alpha}{\pi+2\,\mathrm{SIN}^{-1}(2\alpha-1)}\right]$$

NOTE: $\alpha = \frac{y}{b}$ OR $\frac{y}{D}$

$K = \frac{1.486}{n}AR^{2/3}$ (BAKHMETEFF CONVEYANCE FACTOR)

OPEN CHANNEL FLOW
HYDRAULIC EXPONENT "N"

HYDRAULIC DESIGN CHART 010-4

WES 3-56

η	N											
	2.8	3.0	3.2	3.3	3.4	3.6	3.8	4.0	4.2	4.6	5.0	5.4
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.02	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020
0.04	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040
0.06	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060	0.060
0.08	0.080	0.080	0.080	0.080	0.080	0.080	0.080	0.080	0.080	0.080	0.080	0.080
0.10	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100
0.12	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120	0.120
0.14	0.140	0.140	0.140	0.140	0.140	0.140	0.140	0.140	0.140	0.140	0.140	0.140
0.16	0.160	0.160	0.160	0.160	0.160	0.160	0.160	0.160	0.160	0.160	0.160	0.160
0.18	0.180	0.180	0.180	0.180	0.180	0.180	0.180	0.180	0.180	0.180	0.180	0.180
0.20	0.201	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200
0.22	0.221	0.221	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220
0.24	0.241	0.241	0.241	0.240	0.240	0.240	0.240	0.240	0.240	0.240	0.240	0.240
0.26	0.262	0.261	0.261	0.261	0.261	0.260	0.260	0.260	0.260	0.260	0.260	0.260
0.28	0.282	0.282	0.281	0.281	0.281	0.281	0.280	0.280	0.280	0.280	0.280	0.280
0.30	0.303	0.302	0.302	0.301	0.301	0.301	0.301	0.300	0.300	0.300	0.300	0.300
0.32	0.324	0.323	0.322	0.322	0.322	0.321	0.321	0.321	0.321	0.320	0.320	0.320
0.34	0.344	0.343	0.343	0.342	0.342	0.342	0.341	0.341	0.341	0.340	0.340	0.340
0.36	0.366	0.364	0.363	0.363	0.363	0.362	0.362	0.361	0.361	0.361	0.360	0.360
0.38	0.387	0.385	0.384	0.383	0.383	0.383	0.382	0.382	0.381	0.381	0.381	0.380
0.40	0.408	0.407	0.405	0.404	0.404	0.403	0.403	0.402	0.402	0.401	0.401	0.400
0.42	0.430	0.428	0.426	0.425	0.425	0.424	0.423	0.423	0.422	0.421	0.421	0.421
0.44	0.452	0.450	0.448	0.447	0.446	0.445	0.444	0.443	0.443	0.442	0.441	0.441
0.46	0.475	0.472	0.470	0.469	0.468	0.466	0.465	0.464	0.463	0.462	0.462	0.461
0.48	0.497	0.494	0.492	0.490	0.489	0.488	0.486	0.485	0.484	0.483	0.482	0.481
0.50	0.521	0.517	0.514	0.512	0.511	0.509	0.508	0.506	0.505	0.504	0.503	0.502
0.52	0.544	0.540	0.536	0.534	0.534	0.531	0.529	0.528	0.527	0.525	0.523	0.522
0.54	0.568	0.563	0.559	0.557	0.556	0.554	0.551	0.550	0.548	0.546	0.544	0.543
0.56	0.593	0.587	0.583	0.580	0.579	0.576	0.574	0.572	0.570	0.567	0.565	0.564
0.58	0.618	0.612	0.607	0.604	0.603	0.599	0.596	0.594	0.592	0.589	0.587	0.585
0.60	0.644	0.637	0.631	0.628	0.627	0.623	0.620	0.617	0.614	0.611	0.608	0.606
0.61	0.657	0.650	0.644	0.641	0.639	0.635	0.631	0.628	0.626	0.622	0.619	0.617
0.62	0.671	0.663	0.657	0.653	0.651	0.647	0.643	0.640	0.637	0.633	0.630	0.628
0.63	0.684	0.676	0.669	0.666	0.664	0.659	0.655	0.652	0.649	0.644	0.641	0.638
0.64	0.698	0.690	0.683	0.678	0.677	0.672	0.667	0.664	0.661	0.656	0.652	0.649
0.65	0.712	0.703	0.696	0.692	0.689	0.684	0.680	0.676	0.673	0.667	0.663	0.660
0.66	0.727	0.717	0.709	0.705	0.703	0.697	0.692	0.688	0.685	0.679	0.675	0.672
0.67	0.742	0.731	0.723	0.718	0.716	0.710	0.705	0.701	0.697	0.691	0.686	0.683
0.68	0.757	0.746	0.737	0.732	0.729	0.723	0.718	0.713	0.709	0.703	0.698	0.694
0.69	0.772	0.761	0.751	0.746	0.743	0.737	0.731	0.726	0.722	0.715	0.710	0.706
0.70	0.787	0.776	0.766	0.760	0.757	0.750	0.744	0.739	0.735	0.727	0.722	0.717
0.71	0.804	0.791	0.781	0.775	0.772	0.764	0.758	0.752	0.748	0.740	0.734	0.729
0.72	0.820	0.807	0.796	0.790	0.786	0.779	0.772	0.766	0.761	0.752	0.746	0.741
0.73	0.837	0.823	0.811	0.805	0.802	0.793	0.786	0.780	0.774	0.765	0.759	0.753
0.74	0.854	0.840	0.827	0.820	0.817	0.808	0.800	0.794	0.788	0.779	0.771	0.766

BASIC EQUATION

$$B(\eta) = -\int_0^{\eta} \frac{d\eta}{\eta^{N-1}}$$

WHERE:

$\eta = \frac{y}{y_0}$

N = HYDRAULIC EXPONENT

* FROM TABLES IN BAKHMETEFF'S "HYDRAULICS OF OPEN CHANNEL FLOW." N = 3.3 COMPUTED BY WES.

η = 0.00 TO 0.74 *

OPEN CHANNEL FLOW

VARIED FLOW FUNCTION $B(\eta)$

HYDRAULIC DESIGN CHART 010-5

η	N											
	2.8	3.0	3.2	3.3	3.4	3.6	3.8	4.0	4.2	4.6	5.0	5.4
0.75	0.872	0.857	0.844	0.836	0.833	0.823	0.815	0.808	0.802	0.792	0.784	0.778
0.76	0.890	0.874	0.861	0.853	0.849	0.839	0.830	0.823	0.817	0.806	0.798	0.791
0.77	0.909	0.892	0.878	0.870	0.866	0.855	0.846	0.838	0.831	0.820	0.811	0.804
0.78	0.929	0.911	0.896	0.887	0.883	0.872	0.862	0.854	0.847	0.834	0.825	0.817
0.79	0.949	0.930	0.914	0.905	0.901	0.889	0.879	0.870	0.862	0.849	0.839	0.831
0.80	0.970	0.950	0.934	0.924	0.919	0.907	0.896	0.887	0.878	0.865	0.854	0.845
0.81	0.992	0.971	0.954	0.943	0.938	0.925	0.914	0.904	0.895	0.881	0.869	0.860
0.82	1.015	0.993	0.974	0.964	0.958	0.945	0.932	0.922	0.913	0.897	0.885	0.875
0.83	1.039	1.016	0.996	0.985	0.979	0.965	0.952	0.940	0.931	0.914	0.901	0.890
0.84	1.064	1.040	1.019	1.007	1.001	0.985	0.972	0.960	0.949	0.932	0.918	0.906
0.85	1.091	1.065	1.043	1.030	1.024	1.007	0.993	0.980	0.969	0.950	0.935	0.923
0.86	1.119	1.092	1.068	1.055	1.048	1.031	1.015	1.002	0.990	0.970	0.954	0.940
0.87	1.149	1.120	1.095	1.081	1.074	1.055	1.039	1.025	1.012	0.990	0.973	0.959
0.88	1.181	1.151	1.124	1.109	1.101	1.081	1.064	1.049	1.035	1.012	0.994	0.978
0.89	1.216	1.183	1.155	1.139	1.131	1.110	1.091	1.075	1.060	1.035	1.015	0.999
0.90	1.253	1.218	1.189	1.171	1.163	1.140	1.120	1.103	1.087	1.060	1.039	1.021
0.91	1.294	1.257	1.225	1.206	1.197	1.173	1.152	1.133	1.116	1.088	1.064	1.045
0.92	1.340	1.300	1.266	1.245	1.236	1.210	1.187	1.166	1.148	1.117	1.092	1.072
0.93	1.391	1.348	1.311	1.289	1.279	1.251	1.226	1.204	1.184	1.151	1.123	1.101
0.94	1.449	1.403	1.363	1.339	1.328	1.297	1.270	1.246	1.225	1.188	1.158	1.134
0.95	1.518	1.467	1.423	1.397	1.385	1.352	1.322	1.296	1.272	1.232	1.199	1.172
0.96	1.601	1.545	1.497	1.468	1.454	1.417	1.385	1.355	1.329	1.285	1.248	1.217
0.97	1.707	1.644	1.590	1.558	1.543	1.501	1.464	1.431	1.402	1.351	1.310	1.275
0.975	1.773	1.707	1.649	1.615	1.598	1.554	1.514	1.479	1.447	1.393	1.348	1.311
0.980	1.855	1.783	1.720	1.684	1.666	1.617	1.575	1.536	1.502	1.443	1.395	1.354
0.985	1.959	1.880	1.812	1.772	1.752	1.699	1.652	1.610	1.573	1.508	1.454	1.409
0.990	2.106	2.017	1.940	1.894	1.873	1.814	1.761	1.714	1.671	1.598	1.537	1.487
0.995	2.355	2.250	2.159	2.105	2.079	2.008	1.945	1.889	1.838	1.751	1.678	1.617
0.999	2.931	2.788	2.663	2.590	2.554	2.457	2.370	2.293	2.223	2.101	2.002	1.917

BASIC EQUATION

$$B(\eta) = -\int_0^{\eta} \frac{d\eta}{\eta^{N-1}}$$

WHERE:

$\eta = \frac{y}{y_o}$

N = HYDRAULIC EXPONENT

* FROM TABLES IN BAKHMETEFF'S "HYDRAULICS OF OPEN CHANNEL FLOW." N = 3.3 COMPUTED BY WES.

η = 0.75 TO 0.999*

OPEN CHANNEL FLOW
VARIED FLOW FUNCTION B(η)

HYDRAULIC DESIGN CHART 010-5/1

η	N											
	2.8	3.0	3.2	3.3	3.4	3.6	3.8	4.0	4.2	4.6	5.0	5.4
1.001	2.399	2.184	2.008	1.905	1.856	1.725	1.610	1.508	1.417	1.264	1.138	1.033
1.005	1.818	1.649	1.506	1.422	1.384	1.279	1.188	1.107	1.036	0.915	0.817	0.737
1.010	1.572	1.419	1.291	1.217	1.182	1.089	1.007	0.936	0.873	0.766	0.681	0.610
1.015	1.428	1.286	1.166	1.097	1.065	0.978	0.902	0.836	0.778	0.680	0.602	0.537
1.02	1.327	1.191	1.078	1.013	0.982	0.900	0.828	0.766	0.711	0.620	0.546	0.486
1.03	1.186	1.060	0.955	0.894	0.866	0.790	0.725	0.668	0.618	0.535	0.469	0.415
1.04	1.086	0.967	0.868	0.811	0.785	0.714	0.653	0.600	0.554	0.477	0.415	0.365
1.05	1.010	0.896	0.802	0.747	0.723	0.656	0.598	0.548	0.504	0.432	0.374	0.328
1.06	0.948	0.838	0.748	0.696	0.672	0.608	0.553	0.506	0.464	0.396	0.342	0.298
1.07	0.896	0.790	0.703	0.653	0.630	0.569	0.516	0.471	0.431	0.366	0.315	0.273
1.08	0.851	0.749	0.665	0.617	0.595	0.535	0.485	0.441	0.403	0.341	0.292	0.252
1.09	0.812	0.713	0.631	0.584	0.563	0.506	0.457	0.415	0.379	0.319	0.272	0.234
1.10	0.777	0.681	0.601	0.556	0.536	0.480	0.433	0.392	0.357	0.299	0.254	0.218
1.11	0.746	0.652	0.575	0.531	0.511	0.457	0.411	0.372	0.338	0.282	0.239	0.204
1.12	0.718	0.626	0.551	0.508	0.488	0.436	0.392	0.354	0.321	0.267	0.225	0.192
1.13	0.692	0.602	0.529	0.487	0.468	0.417	0.374	0.337	0.305	0.253	0.212	0.181
1.14	0.669	0.581	0.509	0.468	0.450	0.400	0.358	0.322	0.291	0.240	0.201	0.170
1.15	0.647	0.561	0.490	0.451	0.432	0.384	0.343	0.308	0.278	0.229	0.191	0.161
1.16	0.627	0.542	0.473	0.434	0.417	0.369	0.329	0.295	0.266	0.218	0.181	0.153
1.17	0.608	0.525	0.458	0.419	0.402	0.356	0.317	0.283	0.255	0.208	0.173	0.145
1.18	0.591	0.509	0.443	0.405	0.388	0.343	0.305	0.272	0.244	0.199	0.165	0.138
1.19	0.574	0.494	0.429	0.392	0.375	0.331	0.294	0.262	0.235	0.191	0.157	0.131
1.20	0.559	0.480	0.416	0.380	0.363	0.320	0.283	0.252	0.226	0.183	0.150	0.125
1.22	0.531	0.454	0.392	0.357	0.341	0.299	0.264	0.235	0.209	0.168	0.138	0.114
1.24	0.505	0.431	0.371	0.337	0.322	0.281	0.248	0.219	0.195	0.156	0.127	0.104
1.26	0.482	0.410	0.351	0.319	0.304	0.265	0.233	0.205	0.182	0.145	0.117	0.095
1.28	0.461	0.391	0.334	0.303	0.288	0.250	0.219	0.193	0.170	0.135	0.108	0.088
1.30	0.442	0.373	0.318	0.288	0.274	0.237	0.207	0.181	0.160	0.126	0.100	0.081
1.32	0.424	0.357	0.304	0.274	0.260	0.225	0.196	0.171	0.150	0.118	0.093	0.075
1.34	0.408	0.342	0.290	0.261	0.248	0.214	0.185	0.162	0.142	0.110	0.087	0.069
1.36	0.393	0.329	0.278	0.250	0.237	0.204	0.176	0.153	0.134	0.103	0.081	0.064
1.38	0.378	0.316	0.266	0.238	0.226	0.194	0.167	0.145	0.127	0.097	0.076	0.060
1.40	0.365	0.304	0.256	0.229	0.217	0.185	0.159	0.138	0.120	0.092	0.071	0.056
1.42	0.353	0.293	0.246	0.219	0.208	0.177	0.152	0.131	0.114	0.087	0.067	0.052
1.44	0.341	0.282	0.236	0.211	0.199	0.169	0.145	0.125	0.108	0.082	0.063	0.049
1.46	0.330	0.273	0.227	0.202	0.191	0.162	0.139	0.119	0.103	0.077	0.059	0.046
1.48	0.320	0.263	0.219	0.195	0.184	0.156	0.133	0.113	0.098	0.073	0.056	0.043
1.50	0.310	0.255	0.211	0.188	0.177	0.149	0.127	0.108	0.093	0.069	0.053	0.040
1.55	0.288	0.235	0.194	0.171	0.161	0.135	0.114	0.097	0.083	0.061	0.046	0.035
1.60	0.269	0.218	0.179	0.157	0.148	0.123	0.103	0.087	0.074	0.054	0.040	0.030
1.65	0.251	0.203	0.165	0.145	0.136	0.113	0.094	0.079	0.067	0.048	0.035	0.026
1.70	0.236	0.189	0.153	0.134	0.125	0.103	0.086	0.072	0.060	0.043	0.031	0.023
1.75	0.222	0.177	0.143	0.124	0.116	0.095	0.079	0.065	0.054	0.038	0.027	0.020
1.80	0.209	0.166	0.133	0.116	0.108	0.088	0.072	0.060	0.049	0.034	0.024	0.017
1.85	0.198	0.156	0.125	0.108	0.100	0.082	0.067	0.055	0.045	0.031	0.022	0.015

BASIC EQUATION

$$B(\eta) = -\int_0^{\eta} \frac{d\eta}{\eta^N - 1}$$

WHERE;

$\eta = \frac{y}{y_0}$

N = HYDRAULIC EXPONENT

* FROM TABLES IN BAKHMETEFF'S "HYDRAULICS OF OPEN CHANNEL FLOW." N = 3.3 COMPUTED BY WES.

η = 1.001 TO 1.85*

OPEN CHANNEL FLOW

VARIED FLOW FUNCTION $B(\eta)$

HYDRAULIC DESIGN CHART 010-5/2

η	N 2.8	3.0	3.2	3.3	3.4	3.6	3.8	4.0	4.2	4.6	5.0	5.4
1.90	0.188	0.147	0.117	0.101	0.094	0.076	0.062	0.050	0.041	0.028	0.020	0.014
1.95	0.178	0.139	0.110	0.094	0.088	0.070	0.057	0.046	0.038	0.026	0.018	0.012
2.00	0.169	0.132	0.104	0.089	0.082	0.066	0.053	0.043	0.035	0.023	0.016	0.011
2.1	0.154	0.119	0.092	0.079	0.073	0.058	0.046	0.037	0.030	0.019	0.013	0.009
2.2	0.141	0.107	0.083	0.070	0.065	0.051	0.040	0.032	0.025	0.016	0.011	0.007
2.3	0.129	0.098	0.075	0.063	0.058	0.045	0.035	0.028	0.022	0.014	0.009	0.006
2.4	0.119	0.089	0.068	0.057	0.052	0.040	0.031	0.024	0.019	0.012	0.008	0.005
2.5	0.110	0.082	0.062	0.052	0.047	0.036	0.028	0.022	0.017	0.010	0.006	0.004
2.6	0.102	0.076	0.057	0.047	0.043	0.033	0.025	0.019	0.015	0.009	0.005	0.003
2.7	0.095	0.070	0.052	0.043	0.039	0.029	0.022	0.017	0.013	0.008	0.005	0.003
2.8	0.089	0.065	0.048	0.039	0.036	0.027	0.020	0.015	0.012	0.007	0.004	0.002
2.9	0.083	0.060	0.044	0.036	0.033	0.024	0.018	0.014	0.010	0.006	0.004	0.002
3.0	0.078	0.056	0.041	0.033	0.030	0.022	0.017	0.012	0.009	0.005	0.003	0.002
3.5	0.059	0.041	0.029	0.023	0.021	0.015	0.011	0.008	0.006	0.003	0.002	0.001
4.0	0.046	0.031	0.022	0.017	0.015	0.010	0.007	0.005	0.004	0.002	0.001	0.000
4.5	0.037	0.025	0.017	0.013	0.011	0.008	0.005	0.004	0.003	0.001	0.001	0.000
5.0	0.031	0.020	0.013	0.010	0.009	0.006	0.004	0.003	0.002	0.001	0.000	0.000
6.0	0.022	0.014	0.009	0.007	0.006	0.004	0.002	0.002	0.001	0.000	0.000	0.000
7.0	0.017	0.010	0.006	0.005	0.004	0.002	0.002	0.001	0.001			
8.0	0.013	0.008	0.005	0.003	0.003	0.002	0.001	0.001	0.000			
9.0	0.011	0.006	0.004	0.003	0.002	0.001	0.001	0.000	0.000			
10.0	0.009	0.005	0.003	0.002	0.002	0.001	0.001	0.000	0.000			
20.0	0.006	0.002	0.001	0.001	0.001	0.000	0.000	0.000	0.000			

BASIC EQUATION

$$B(\eta) = -\int_0^{\eta} \frac{d\eta}{\eta^{N}-1}$$

WHERE:

$\eta = \frac{y}{y_0}$

N = HYDRAULIC EXPONENT

* FROM TABLES IN BAKHMETEFF'S "HYDRAULICS OF OPEN CHANNEL FLOW." N = 3.3 COMPUTED BY WES.

η = 1.90 TO 20.0*

OPEN CHANNEL FLOW
VARIED FLOW FUNCTION B(η)

HYDRAULIC DESIGN CHART 010-5/3

WES 3-56

HYDRAULIC DESIGN CRITERIA

SHEETS 010-6 TO 010-6/5

OPEN CHANNEL FLOW

BRIDGE PIER LOSSES

Background

1. Methods for computing head losses at bridge piers have been developed by D'Aubuisson, Nagler, Yarnell, Koch and Carstanjen, and others. Each method is based on experimental data for limited flow conditions. Complete agreement between methods is not always obtained. The energy method of Yarnell(4) and the momentum method of Koch and Carstanjen(1) have been widely used in the United States.

Equations for Classes of Flow

2. Three classes of flow conditions, A, B, and C, are encountered in the bridge pier problem. Hydraulic Design Chart 010-6 illustrates the flow condition upstream from, within, and downstream from the bridge section for each class of flow. The energy method of Yarnell is generally used for the solution of Class A flow problems, and is also used for solution of Class B flow. However, the momentum method of Koch and Carstanjen is believed more applicable to Class B flow, and is also applicable for solution of Class C flow.

3. Energy Method, Class A Flow. The Yarnell equation for Class A flow is

$$H_3 = 2K(K + 10\omega - 0.6)(\alpha + 15\alpha^4)\frac{V_3^2}{2g}$$

where

H_3 = drop in water surface, in ft, from upstream to downstream at the contraction

K = experimental pier shape coefficient

ω = ratio of velocity head to depth downstream from the contraction

α = horizontal contraction ratio

V_3 = velocity downstream from the contraction in ft per sec

g = acceleration, gravitational, in ft per sec^2

The values of K determined by Yarnell for different pier shapes are

Pier Shape	K
Semicircular nose and tail	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90 deg triangular nose and tail	1.05
Square nose and tail	1.25

4. Energy Method, Class B Flow. The Yarnell equations for Class B flow are

$$L_B = C_B \frac{V_1^2}{2g}$$

and

$$C_B = 0.50 + K_B(5.5\alpha^3 + 0.08)$$

where

L_B = pier nose loss in ft

C_B = pier nose loss coefficient

V_1 = velocity upstream from the contraction in ft per sec

K_B = experimental pier shape coefficient

The values of K_B determined by Yarnell for different pier shapes are

Pier Shape	K_B
Square nose piers	5
Round nose piers	1

The following equation permits solution of the Yarnell equation for Class B flow by successive approximation

$$d_1 = d_L + L_B$$

where

d_1 = upstream water depth in ft

d_L = the higher depth, in ft, in the unobstructed channel which has flow of equal energy to that required for critical flow within the constricted bridge section

5. Momentum Method, Class B Flow. Koch and Carstanjen applied the momentum principle to flow past bridge piers and verified their results by laboratory investigations. The total upstream momentum minus the momentum loss at the entrance equals the total momentum within the pier section. This momentum quantity is also equal to the total momentum downstream minus the static pressure on the downstream obstructed area. The general momentum equation is

$$m_1 - m_p + \frac{\gamma Q^2}{gA_1^2}(A_1 - A_p) = m_2 + \frac{\gamma Q^2}{gA_2} = m_3 - m_p + \frac{\gamma Q^2}{gA_3}$$

where

Q = discharge in cfs

m_1, m_2, m_3, m_p = total static pressure of water in the upstream section, pier section, downstream section, and on the pier ends, respectively, in lb

A_1, A_p, A_2, A_3 = cross-sectional area of the upstream channel, pier obstruction, channel within the pier section, and downstream channel, respectively, in sq ft

γ = specific weight of water, 62.5 lb/cu ft

6. Graphical Solutions. The U. S. Army Engineer District, Los Angeles(3), modified Yarnell's charts for solution of Class A and Class B flow, and developed a graphical solution for Class B flow by the momentum method. The U. S. Army Engineer District, Chicago (2), simplified the Los Angeles District's graphical solution for Class B flow by the energy method. Hydraulic Design Charts 010-6/2 and 010-6/3, respectively, present the Los Angeles District solutions for Class A flow by the energy method and Class B flow by the momentum method. Chart 010-6/4 presents the Chicago District's solution for Class B flow by the energy method.

Application

7. Classification of Flow. Flow classification can be determined from Chart 010-6/1. The intersection of the computed value of λ (the ratio of the channel depth without piers to the critical depth) and α (the horizontal contraction ratio) determines the flow classification.

8. Class A Flow. Chart 010-6/2 presents a graphical solution of Class A flow for five types of bridge piers. Enter the chart horizontally with a known λ_3 to a known α. Determine the value of X. The head loss through the pier section (H_3) is obtained by multiplying the critical depth in the unobstructed channel by X for round nose piers or by γX for the other pier shapes shown on the chart.

9. Class B Flow. Bridge pier losses by the momentum method can be determined from Chart 010-6/3. For a known value of α, the required ratio of d_1/d_c can be obtained and the upstream depth computed. Chart 010-6/4 permits solution of Class B flow for round and square nose piers by the energy method. This chart is used in the same manner as Chart 010-6/3.

10. Class C Flow. Class C flow is seldom encountered in practical problems. A graphical solution has not been developed, and

analytical solution by the momentum method is necessary.

11. Sample Computation. Chart 010-6/5 is a sample computation illustrating the use of the charts. A borderline flow condition between Class A and Class B is assumed. This permits three solutions to the problem. The most conservative solution is recommended for design purposes.

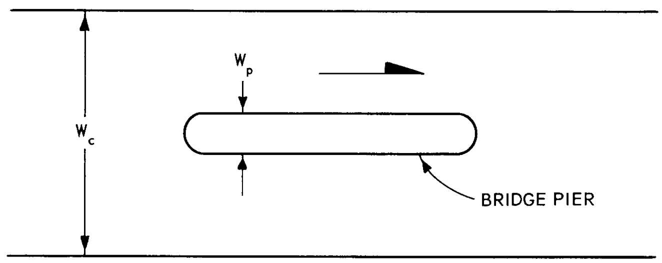
12. References.

(1) Koch, A., Von der Bewegung des Wassers und den dabei auftretenden Kräften, M. Carstanjen, ed. Julius Springer, Berlin, 1926.

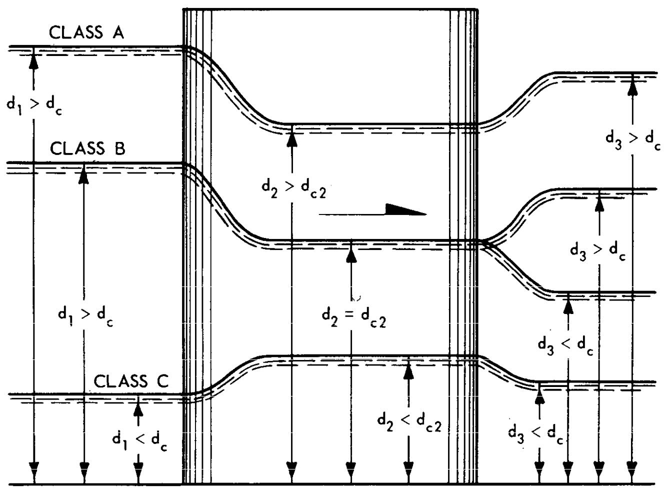
(2) U. S. Army Engineer District, Chicago, CE, letter to U. S. Army Engineer Division, Great Lakes, CE, dated 22 April 1954, subject, "Analysis of Flows in Channels Constricted by Bridge Piers."

(3) U. S. Army Engineer District, Los Angeles, CE, Report on Engineering Aspects, Flood of March 1938, Appendix I, Theoretical and Observed Bridge Pier Losses. Los Angeles, Calif., May 1939.

(4) Yarnell, David L., Bridge Piers as Channel Obstructions. U. S. Department of Agriculture Technical Bulletin No. 442, Washington, D. C., November 1934.



PLAN



ELEVATION

NOTE: $\alpha = W_p/W_c$ = HORIZONTAL CONTRACTION RATIO

W_p = TOTAL PIER WIDTH

W_c = GROSS CHANNEL WIDTH

d_1 = UPSTREAM DEPTH

d_2 = DEPTH WITHIN PIER SECTION

d_3 = DOWNSTREAM DEPTH

d_c = CRITICAL DEPTH WITHIN THE UNOBSTRUCTED CHANNEL SECTION

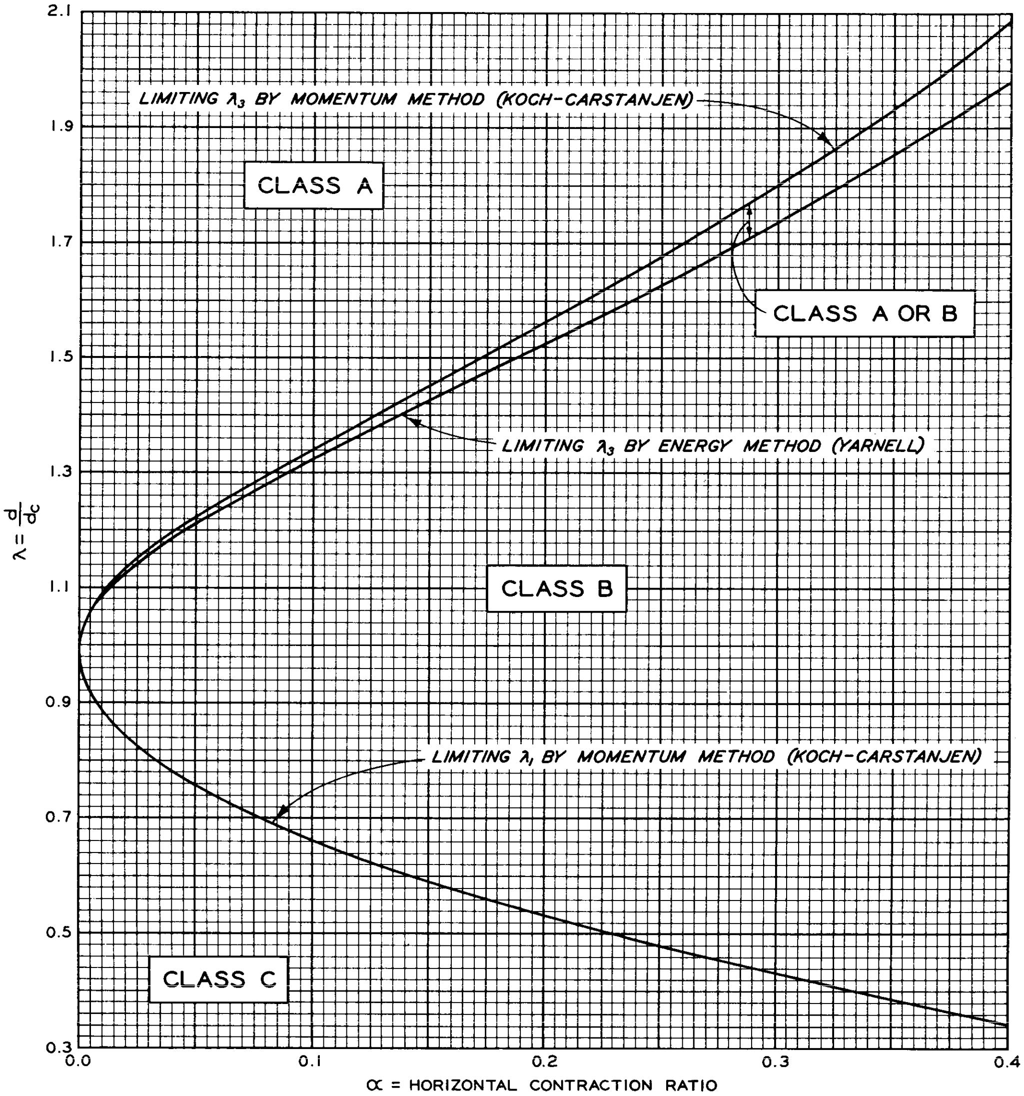
d_{c2} = CRITICAL DEPTH WITHIN THE PIER SECTION

OPEN CHANNEL FLOW

RECTANGULAR SECTION
BRIDGE PIER LOSSES
DEFINITION

HYDRAULIC DESIGN CHART 010-6

WES 5-59



<u>EQUATIONS FOR LIMITING λ</u>

λ_3 - ENERGY METHOD (YARNELL)

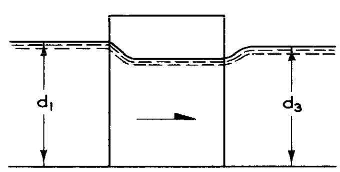
$$\alpha = 1 - \left[\frac{3\lambda_3^2}{2\lambda_3^3 + 1}\right]^{3/2}$$

λ_3 - MOMENTUM METHOD (KOCH-CARSTANJEN)

$$\alpha = 1 - \left[\frac{3\lambda_3}{(1-\alpha)\lambda_3^3 + 2}\right]^{3}$$

λ_1 - MOMENTUM METHOD (KOCH-CARSTANJEN)

$$\alpha = 1 - \left[\frac{3\lambda_1}{\lambda_1^3 + 2}\right]^{3/4}$$



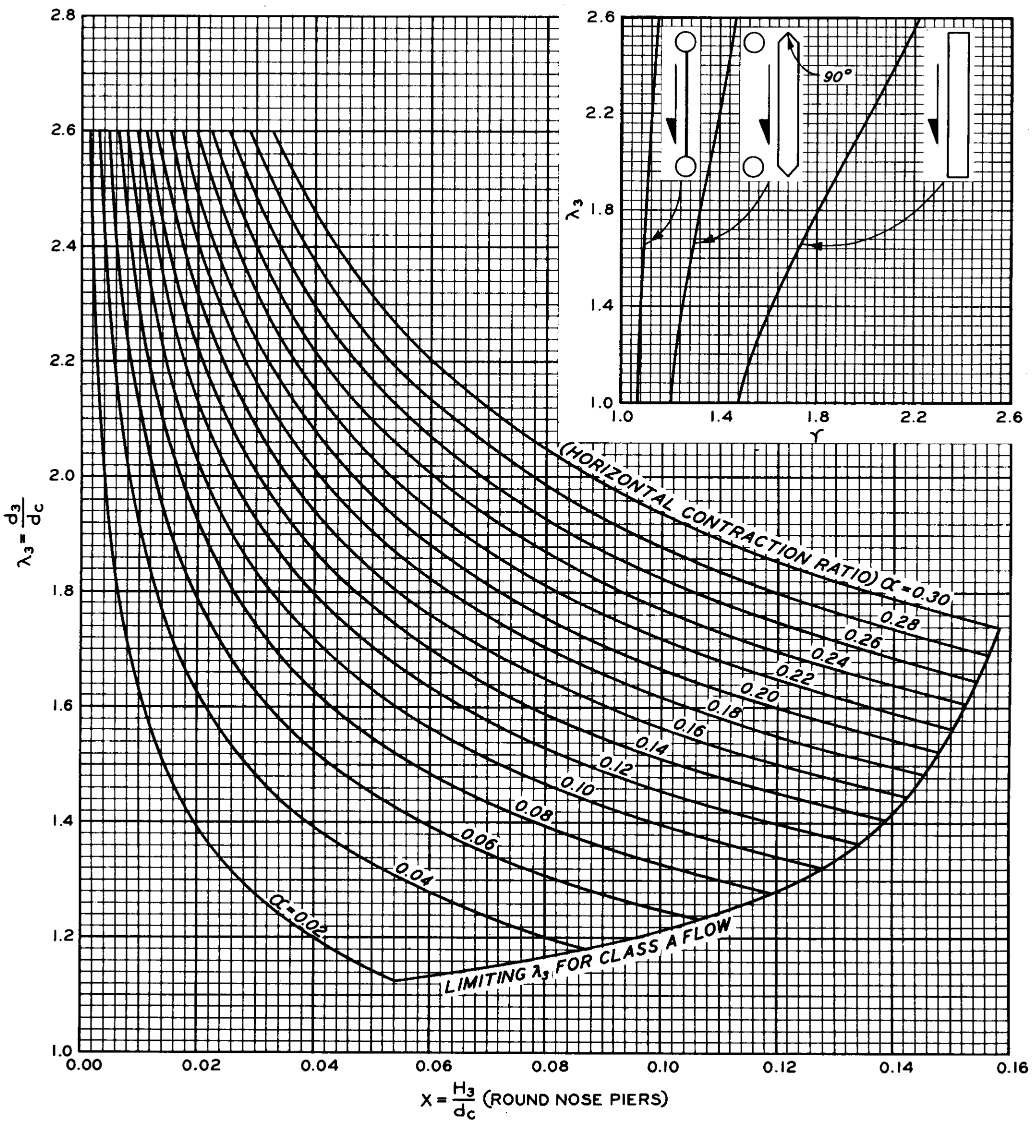
<u>DEFINITION SKETCH</u>

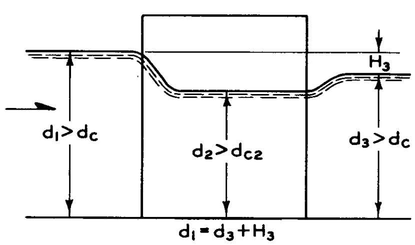
NOTE: $\lambda_1 = d_1/d_c$
$\lambda_3 = d_3/d_c$
d_1 = UPSTREAM WATER DEPTH
d_3 = DOWNSTREAM WATER DEPTH
d_c = CRITICAL DEPTH WITHIN THE UNOBSTRUCTED CHANNEL SECTION
α = HORIZONTAL CONTRACTION RATIO (Σ PIER WIDTHS $\div$ CHANNEL WIDTH)
d = DEPTH WITHOUT BRIDGE PIERS

OPEN CHANNEL FLOW

RECTANGULAR SECTION
BRIDGE PIER LOSSES
CLASSIFICATION OF FLOW CONDITIONS

HYDRAULIC DESIGN CHART 010-6/1





DEFINITION SKETCH

NOTE: d_c = CRITICAL DEPTH WITHIN THE UNOBSTRUCTED CHANNEL SECTION

d_{c2} = CRITICAL DEPTH WITHIN THE PIER SECTION

H_3 = Xd_c (ROUND NOSE PIERS)

H_3 = $Xd_c\Upsilon$ (INDICATED SHAPES)

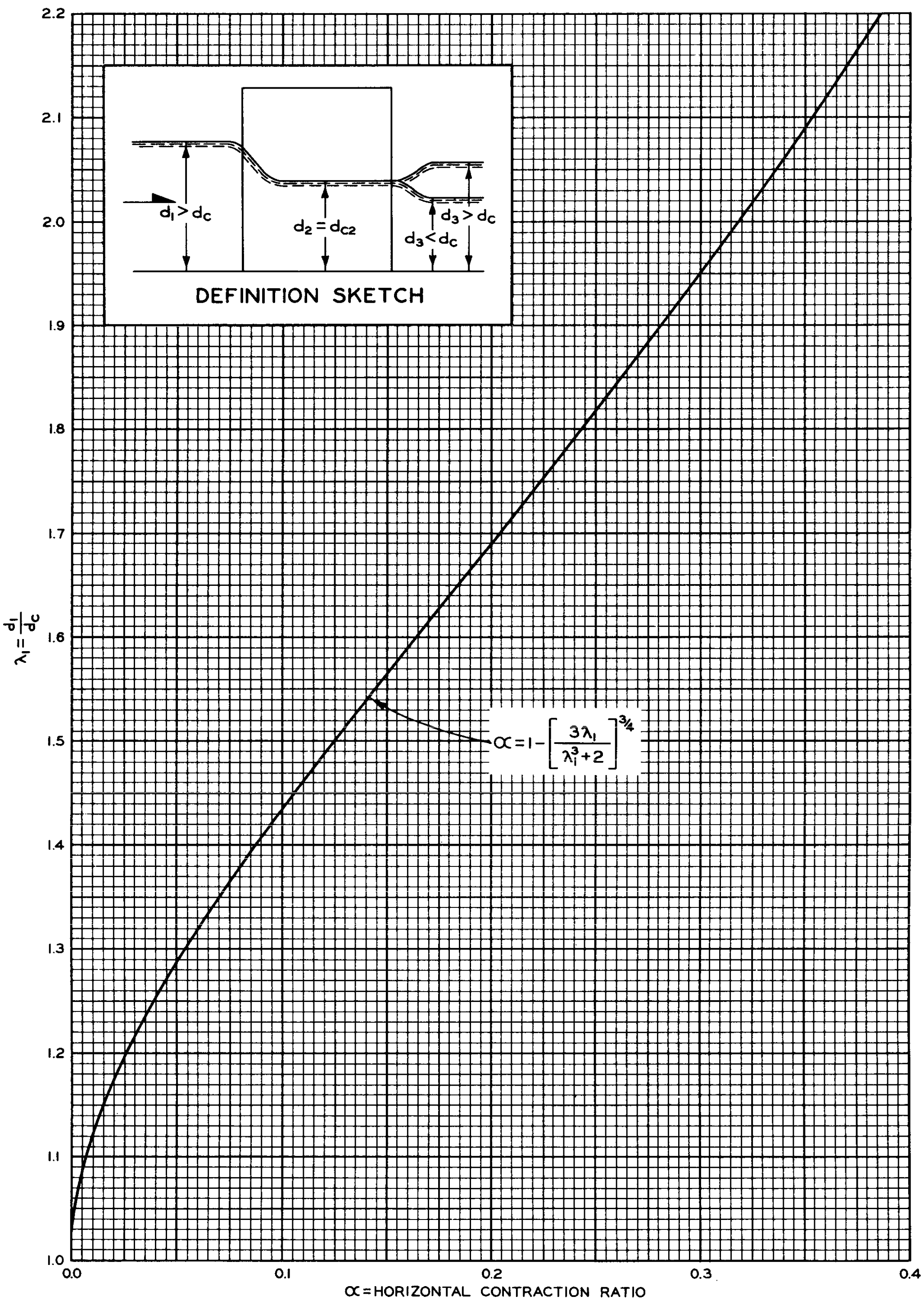
OPEN CHANNEL FLOW

RECTANGULAR SECTION
BRIDGE PIER LOSSES
CLASS A FLOW – ENERGY METHOD

HYDRAULIC DESIGN CHART 010-6/2

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 5-59



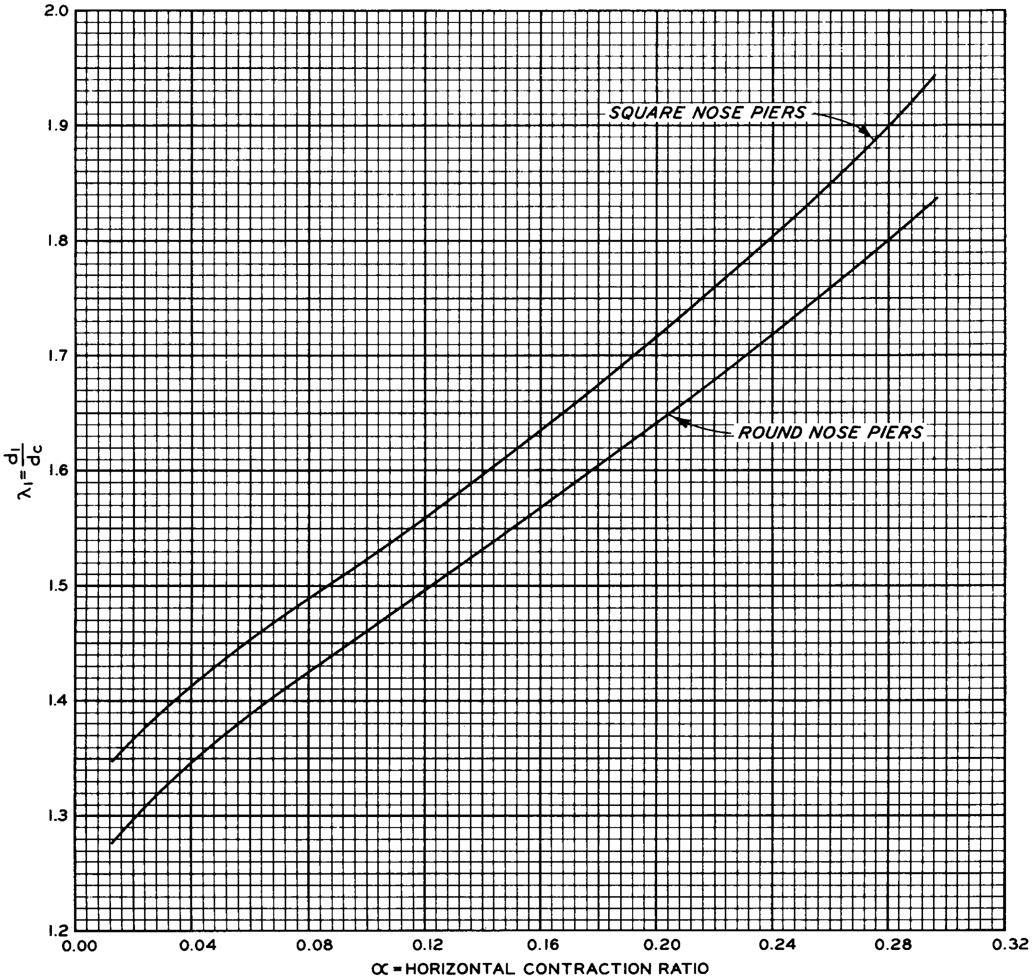
NOTE: λ_1 = d_1/d_c
d_1 = UPSTREAM WATER DEPTH
d_c = CRITICAL DEPTH WITHIN THE UNOBSTRUCTED CHANNEL SECTION
d_{c2} = CRITICAL DEPTH WITHIN THE PIER SECTION
α = HORIZONTAL CONTRACTION RATIO

OPEN CHANNEL FLOW
RECTANGULAR SECTION
BRIDGE PIER LOSSES
CLASS B FLOW – MOMENTUM METHOD

HYDRAULIC DESIGN CHART 010-6/3

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 5-59

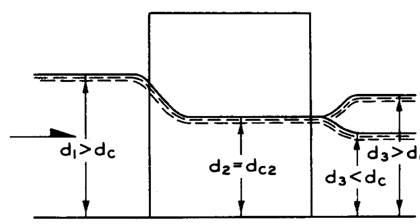


EQUATIONS

$$\frac{1}{(1-\alpha)^{2/3}} = \frac{1}{3\lambda_L^2} + \frac{2\lambda_L}{3}$$

$$\lambda_1 = \lambda_L + \frac{0.5 + K_B(5.5\,\alpha^3 + 0.08)}{2\lambda_1^2}$$

NOTE: λ_1 = d_1/d_c
λ_3 = d_3/d_c
λ_L = LIMITING λ_3 BY ENERGY METHOD
d_1 = UPSTREAM WATER DEPTH
d_3 = DOWNSTREAM WATER DEPTH
d_c = CRITICAL DEPTH WITHIN THE UNOBSTRUCTED CHANNEL SECTION
d_{c2} = CRITICAL DEPTH WITHIN THE PIER SECTION
α = HORIZONTAL CONTRACTION RATIO
K_B = YARNELL PIER-SHAPE COEFFICIENT
(1.0 FOR ROUND NOSE)
(5.0 FOR SQUARE NOSE)



DEFINITION SKETCH

OPEN CHANNEL FLOW
RECTANGULAR SECTION
BRIDGE PIER LOSSES
CLASS B FLOW – ENERGY METHOD

HYDRAULIC DESIGN CHART 010-6/4

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 5-59

U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION
COMPUTATION SHEET

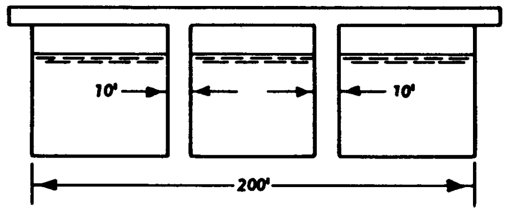
JOB CW 804 PROJECT John Doe River SUBJECT Rectangular Channel

COMPUTATION Bridge Pier Loss

COMPUTED BY MBB DATE 12/17/58 CHECKED BY WTH DATE 12/18/58

GIVEN:

Rectangular channel section
Round nose piers
Channel discharge (Q) = 40,000 cfs
Channel width (W_c) = 200 ft
Total pier width (W_p) = 20 ft
Depth without bridge piers (d) = 14.3 ft



COMPUTE:

1. Horizontal contraction ratio (α)

$$\alpha = \frac{W_p}{W_c} = \frac{20}{200} = 0.10$$

2. Discharge (q) per ft of channel width

$$q = \frac{Q}{W_c} = \frac{40{,}000}{200} = 200 \text{ cfs}$$

3. Critical depth (d_c) in unobstructed channel
   From Chart 610-8, d_c = 10.8 ft
   for q = 200 cfs.

4. $\lambda = d/d_c = 14.3/10.8$
   $= 1.324$

5. Flow classification
   On Chart 010-6/1, intersection of α = 0.10 and λ = 1.324 is in zone marked Class A or B.

6. Upstream depth (d_1)
   a. Class A flow - Energy Method

      $d_1 = d_3 + H_3$ (Chart 010-6/2

      $H_3 = Xd_c$

      $X = 0.127$ for $\alpha = 0.10$
      and $\lambda_3 = \lambda = 1.324$

      $H_3 = 0.127 \times 10.8 = 1.37$

      $d_1 = 14.3 + 1.37 = 15.67$ ft

   b. Class B flow - Momentum Method

      $d_1 = \lambda_1 d_c$ (Chart 010-6/3)
      $\lambda_1 = 1.435$ for $\alpha = 0.10$

      $d_1 = 1.435 \times 10.8 = 15.50$ ft

   c. Class B flow - Energy Method

      $d_1 = \lambda_1 d_c$ (Chart 010-6/4)
      $\lambda_1 = 1.460$ for $\alpha = 0.10$

      $d_1 = 1.460 \times 10.8 = 15.77$ ft

OPEN CHANNEL FLOW
RECTANGULAR SECTION
BRIDGE PIER LOSSES
SAMPLE COMPUTATION
HYDRAULIC DESIGN CHART 010-6/5

SHEET 010-7

OPEN CHANNEL FLOW

TRASH RACK LOSSES

1. The energy loss of flow through trash racks depends upon the shape, size, and spacing of the bars and the velocity of flow. Hydraulic Design Chart 010-7 shows loss coefficient curves for different bar designs. The curves are based on tests in open channels with the racks perpendicular to the line of flow.

2. Stockholm Tests. Tests made in the Hydraulic Structures Laboratory of the Royal Technical University at Stockholm, Sweden, were reported by W. Fellenius(1). The publication also presents results for bar shapes and sizes not included on Chart 010-7. The effects of sloping the racks were also studied.

3. Munich Tests. Tests made in the Hydraulic Institute of the Technical University at Munich, Germany, were reported by O. Kirschmer(2). The tests included other bar shapes not shown on the chart. The effects of tilting the rack were also studied. Spangler(3) investigated the effects of varying the horizontal angle of approach channel to the trash rack.

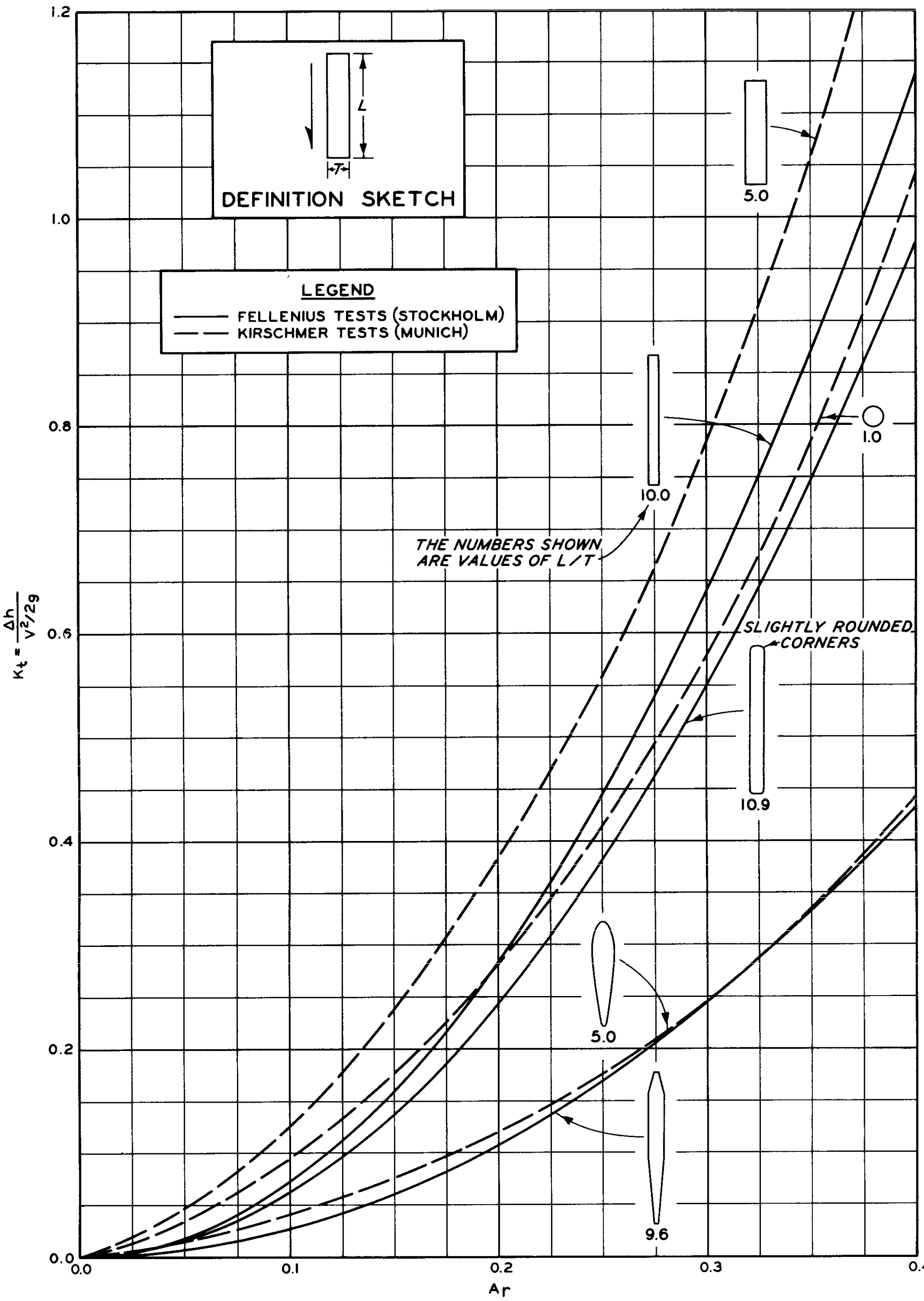
4. Application. The loss coefficients shown on Chart 010-7 were obtained from tests in which the racks protruded above the water surface. The applicability of the data to submerged racks is not known. As stated above, numerous other shapes were tested at Stockholm and Munich. The data presented on Chart 010-7 were selected to demonstrate the general effect of bar shape on head loss.

5. References.

(1) Fellenius, W., "Experiments on the head loss caused by protecting racks at water-power plants." Meddelande No. 5 Vattenbyggnadsinstitutionen, Vid Kungl. Tekniska Hogskolan, Stockholm (1928). Summary and pertinent data also published in Hydraulic Laboratory Practice, ASME (1929), p 533.

(2) Kirschmer, O., "Investigation regarding the determination of head loss." Mitteilungen des Hydraulischen Instituts der Technischen Hochschule Munchen, Heft 1 (1926), p 21.

(3) Spangler, J., "Investigations of the loss through trash racks inclined obliquely to the stream flow." Mitteilungen des Hydraulischen Instituts der Technischen Hochschule Munchen, Heft 2 (1928), p 46. English translation published in Hydraulic Laboratory Practice, ASME (1929), p 461.



NOTE: Δh = HEAD LOSS THROUGH RACK IN FT
V = VELOCITY AT SECTION WITHOUT RACK IN FT/SEC
K_t = HEAD LOSS COEFFICIENT
$A_r = \frac{\text{AREA OF BARS}}{\text{AREA OF SECTION}}$

OPEN CHANNEL FLOW

TRASH RACK LOSSES

HYDRAULIC DESIGN CHART 010-7

PREPARED BY U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION, VICKSBURG, MISSISSIPPI

WES 5-59

AIR DEMAND - REGULATED OUTLET WORKS

1. Background. The data presented are considered applicable to slide and tractor gates operating in rectangular gate chambers. Previous designs of air vents have been based on arbitrary adoption of a ratio of the cross-sectional area of the air vent to that of the conduit being aerated.

2. Iowa Tests. Kalinske and Robertson* have published the results of tests on the air demand of a hydraulic jump in a circular conduit. They found the ratio of air demand to water discharge (β) to be a function of the Froude number minus one. The formula which was developed is indicated in HDC 050-1.

3. Prototype Tests. A number of prototype tests on existing outlet works have been analyzed and compared graphically with the Kalinske and Robertson formula in HDC 050-1. In some of the prototype tests, gate openings varied from small to full opening where pressure flow existed throughout the entire system. The maximum air demand is found at some intermediate gate opening. The ratios of this gate opening (G_m) to full gate opening (G_f) are shown in table 1 together with other pertinent information.

Table 1

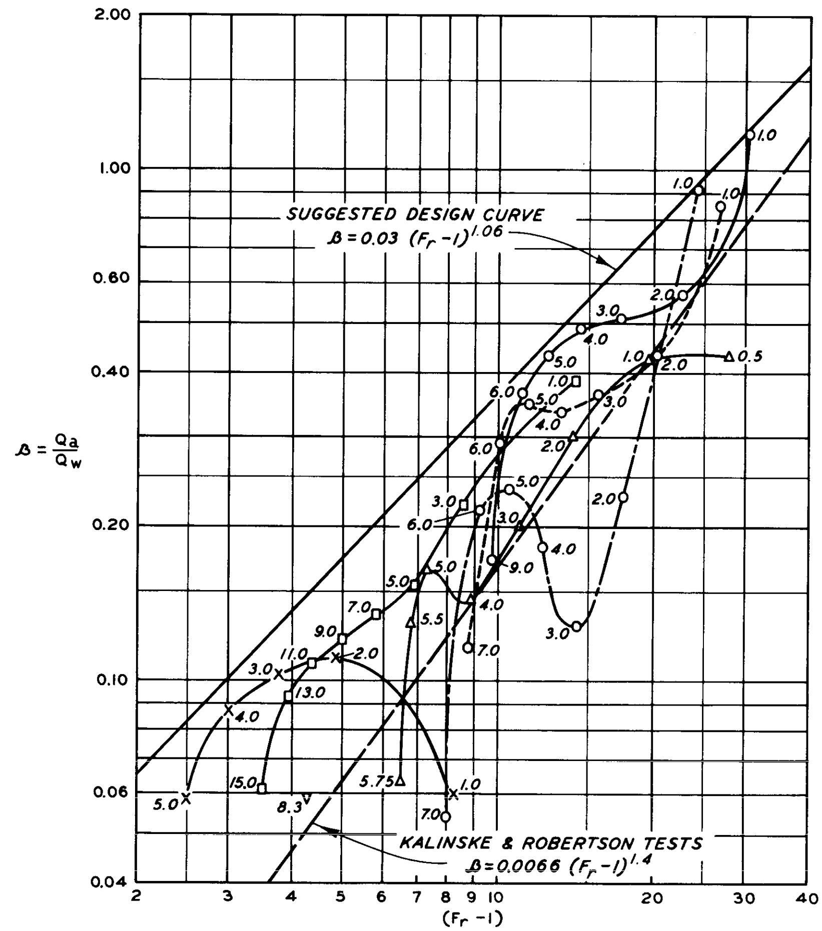
Dam	Max Air Velocity ft/sec	Vent Area A_v sq ft	Conduit Area A_c sq ft	$\frac{A_v}{A_c}$	Gate Openings ft		$\frac{G_m}{G_f}$
					Max Air G_m	Full G_f	
Pine Flat	280	4.91	45.0	0.109	5.5	9.0	0.611
Tygart	219	0.79	56.7	0.014	8.3	10.0	0.833
Norfork	127	2.18	24.0	0.091	5.0	6.0	0.833
Denison	57	22.33	314.2	0.071	13.0	19.0	0.685
Hulah	36	1.40	32.5	0.043	4.0	6.5	0.615

4. Extensive Corps of Engineers air-demand tests were made at Pine Flat Dam from 1952 to 1956. These tests included heads up to 370 ft

* A. A. Kalinske and J. W. Robertson, "Entrainment of air in flowing water--closed conduit flow." Transactions, American Society of Civil Engineers, vol 108 (1943), pp 1435-1447.

although gates are not normally operated under such high heads. The Pine Flat test data are in good agreement with other field data, as shown by the plots in HDC 050-1.

5. Recommendations. A straight line in HDC 050-1 indicates a suggested design assumption. It is suggested that the maximum air demand be assumed to occur at a gate opening ratio of 80 percent in sluices through concrete dams. A gate lip with a 45-degree angle on the bottom can be expected to have a contraction coefficient of approximately 0.80. The Froude number should be based on the effective depth at the vena contracta which, with the above-mentioned factors, would be 64 percent of the sluice depth. The suggested design curve can be used to determine the ratios of air demand to water discharge. It is further suggested that air vents be designed for velocities of not more than 150 ft per sec. The disadvantage of excessive air velocities is a high head loss in the air vent which causes subatmospheric pressures in the water conduit. Outlet works with well-streamlined water passages can tolerate lower pressures without cavitation trouble than those with less effectively streamlined water passages. The suggested design assumptions for sluices will result in area ratios of air vent to sluice of approximately 12 percent for each 150 ft of head on a 4- by 6-ft sluice, and 12 percent for each 200 ft of head on a 5-ft-8-in. by 10-ft sluice. In applying the curve to circular tunnels controlled by one or more rectangular gates, the effective depth should be based on flow in 64 percent of the area of the tunnel for maximum air demand. These are general design rules which have been devised until additional experimental data are available.



NOTE : $F_r = V/\sqrt{gy}$ (FROUDE NUMBER)

V = WATER VELOCITY AT VENA CONTRACTA, FT/SEC

y = WATER DEPTH AT VENA CONTRACTA, FT

Q_a = AIR DEMAND, CFS

Q_w = WATER DISCHARGE, CFS

LEGEND

O———O PINE FLAT - H = 370 FT

O------O PINE FLAT - H = 304 FT

O—-—-O PINE FLAT - H = 254 FT

□———□ DENISON - H = 84 FT

X———X HULAH - H = 24 FT

Δ———Δ NORFORK - H = 154 FT

∇ TYGART - H = 92 FT

H = HEAD, POOL TO CONDUIT CENTER LINE

FIGURES ON GRAPH SHOW GATE OPENING IN FEET.

AIR DEMAND
REGULATED OUTLET WORKS

HYDRAULIC DESIGN CHART 050-1

REV 1-64 WES 4-1-52

SHEET 050-1/1

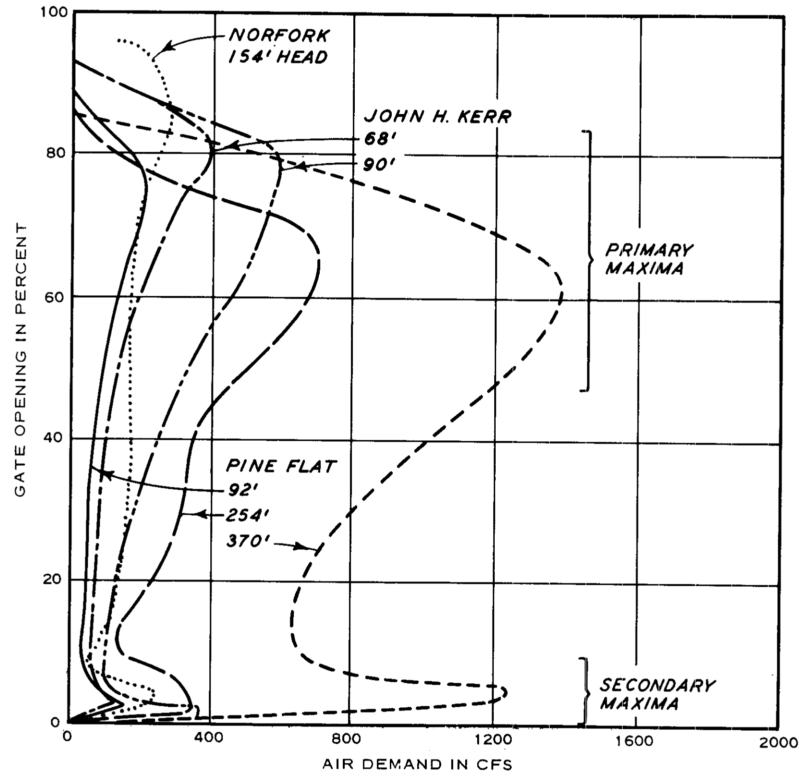
AIR DEMAND - REGULATED OUTLET WORKS

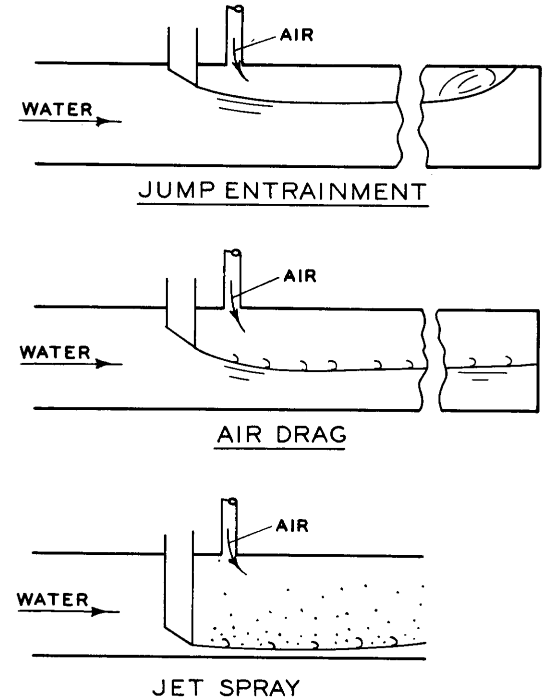
PRIMARY AND SECONDARY MAXIMA

1. Field tests to determine air demand in regulated outlet works have indicated two gate positions at which the air demand greatly exceeds that of other gate openings. Large quantities of air are required when the gate is about 5 per cent open and again at some gate position between 50 and 100 per cent open. Hydraulic Design Chart 050-1/1 shows the observed air demand in cfs plotted against per cent of gate opening for a number of operating heads at Pine Flat, Norfork, and John H. Kerr Dams. The chart also indicates flow conditions below the gate for various openings.

2. At small gate openings the jet frays or breaks up and entrains large quantities of air. As the gate opening increases the air demand rapidly decreases and then increases to a second maximum just before the conduit flows full at the exit portal. In this phase of operation, the air demand is caused by the drag force between the water surface and the air above. With larger gate openings, a hydraulic jump forms in the conduit and the air demand is limited by the capacity of the jump to entrain and remove air. When the conduit flows full the air demand becomes zero.

3. Chart 050-1/1 is included to show the qualitative characteristics of air demand. Sufficient prototype data are not available to develop a relationship between air demand, head, and other factors.





	CONDUIT		AIR VENT	
	LENGTH, FT	SIZE, FT	LENGTH, FT	DIAMETER, FT
JOHN H. KERR	117	5.67 × 10	90	2.50
NORFORK	195	4 × 6	188	1.67
PINE FLAT	334	5 × 9	320	2.50

NOTE: HEADS ARE MEASURED FROM POOL TO CENTER LINE OF CONDUIT.

AIR DEMAND

REGULATED OUTLET WORKS PRIMARY AND SECONDARY MAXIMA

HYDRAULIC DESIGN CHART 050-1/1

REV 1-64 WES 5-59

SHEET 050-2

SAMPLE AIR VENT DESIGN COMPUTATIONS

1. A sample computation for the design of an air vent is given on Hydraulic Design Chart 050-2. This computation is included in order to clarify the explanation given on sheet 050-1. The coefficients of discharge as given on Chart 320-1 may be considered to be contraction coefficients for determining the depth of water at the vena contracta.

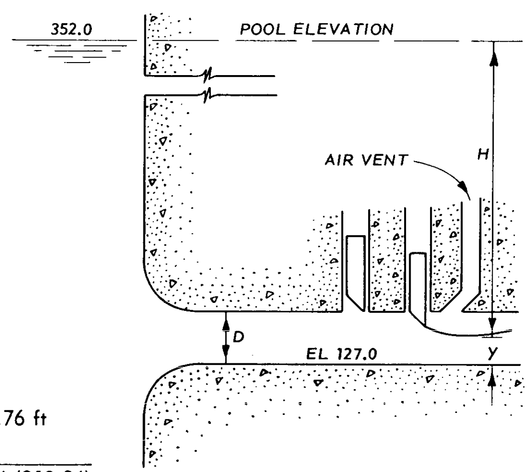
U. S. ARMY ENGINEER WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB: ES 804 PROJECT: John Doe Dam SUBJECT: Air Demand

COMPUTATION: Air Vent Size, Hydraulic Design

COMPUTED BY: BG DATE: 9/5/52 CHECKED BY: AAMC DATE: 9/5/52



GIVEN:

Sluice size: Width (B) = 4 ft
Height (D) = 9 ft
45° gate lip

Elevation sluice invert at gate 127.0

Design pool elevation 352.0

FROM HYDRAULIC DESIGN SHEET 050-1 AND 320-1

Assume maximum air discharge (Q_a) at 80% gate opening.
Discharge coefficient (C) for 45° gate lip = 0.80.

Then:

Depth of water at vena contracta (y) = 80% × 0.80 × 9.0 = 5.76 ft

Effective head, H = 352.0 − (127.0 + 5.76) = 219.24

Water discharge $(Q_w) = CAV = By\sqrt{2gH} = 4.0\,(5.76)\,\sqrt{64.4\,(219.24)}$

$Q_w = 2740$ cfs

$V = \frac{Q_w}{A} = \frac{2740}{4.0 \times 5.76} = 119.0$ ft/sec velocity of water at vena contracta

$F = \frac{V}{\sqrt{gy}} = \frac{119.0}{\sqrt{32.2\,(5.76)}} = 8.75$ Froude number at vena contracta

$(F - 1) = 7.75$

FROM HYDRAULIC DESIGN CHART 050-1. $\beta = 0.28$

$Q_a = \beta Q_w = 0.28\,(2740)$

$Q_a = 767$ cfs

FROM HYDRAULIC DESIGN SHEET 050-1. Maximum Air Velocity (V_a) = 150 ft/sec

$A_v = \frac{767}{150} = 5.1$ sq ft area of air vent required

Diameter for circular vent = 2.55 ft

AIR DEMAND
REGULATED OUTLET WORKS
SAMPLE COMPUTATION

HYDRAULIC DESIGN CRITERIA

SHEET 050-3

AIR ENTRAINMENT

WIDE CHUTE FLOW

1. Purpose. The entrainment of air in flow through a chute spillway causes bulking which necessitates increasing the sidewall design height. HDC 050-3 may be used to estimate the percentage by volume of air that will be entrained in the flow at terminal velocity and its effect on flow depth.

2. Previous Criteria. Previous criteria for estimating air entrainment have been influenced by investigations on narrow chutes as reported by Hall.(2) The data for flows through narrow chutes show the marked effects of sidewalls on the amount of air entrained.

3. Basic Data. Recent tests at the University of Minnesota(3) on artificially roughened channels have afforded new information on chutes for relatively large width-depth ratios which eliminate the sidewall effect. It was found that the mean air concentration ratio of air volume to air-plus-water volume, $\overline{C}$, is a function of the shear velocity and transition depth parameter, $V_s/(d_T)^{2/3}$. This suggests that the intensity of the turbulent fluctuations causing air entrainment is increasingly damped with increasing depth. A more convenient empirical expression of this parameter is the ratio of the sine of the bottom slope to the unit discharge in cubic feet per second, $S/q^{1/5}$. This ratio is used in HDC 050-3.

4. The Minnesota laboratory data are shown in HDC 050-3 together with field data for the Kittitas chute.(2) The Minnesota data were obtained by the use of highly refined electronic equipment developed at St. Anthony Falls Hydraulic Laboratory, University of Minnesota. The Kittitas results were derived from field measurements of water-surface elevations under conditions of high velocities and great turbulence. The Kittitas data selected for use in developing HDC 050-3 were for flows with flow width exceeding five times the depth to eliminate the sidewall effect. The low concentration ratios indicated by these data appear consistent with visual observations of flows near the downstream ends of the Fort Peck(1) and Arkabutla spillway chutes.

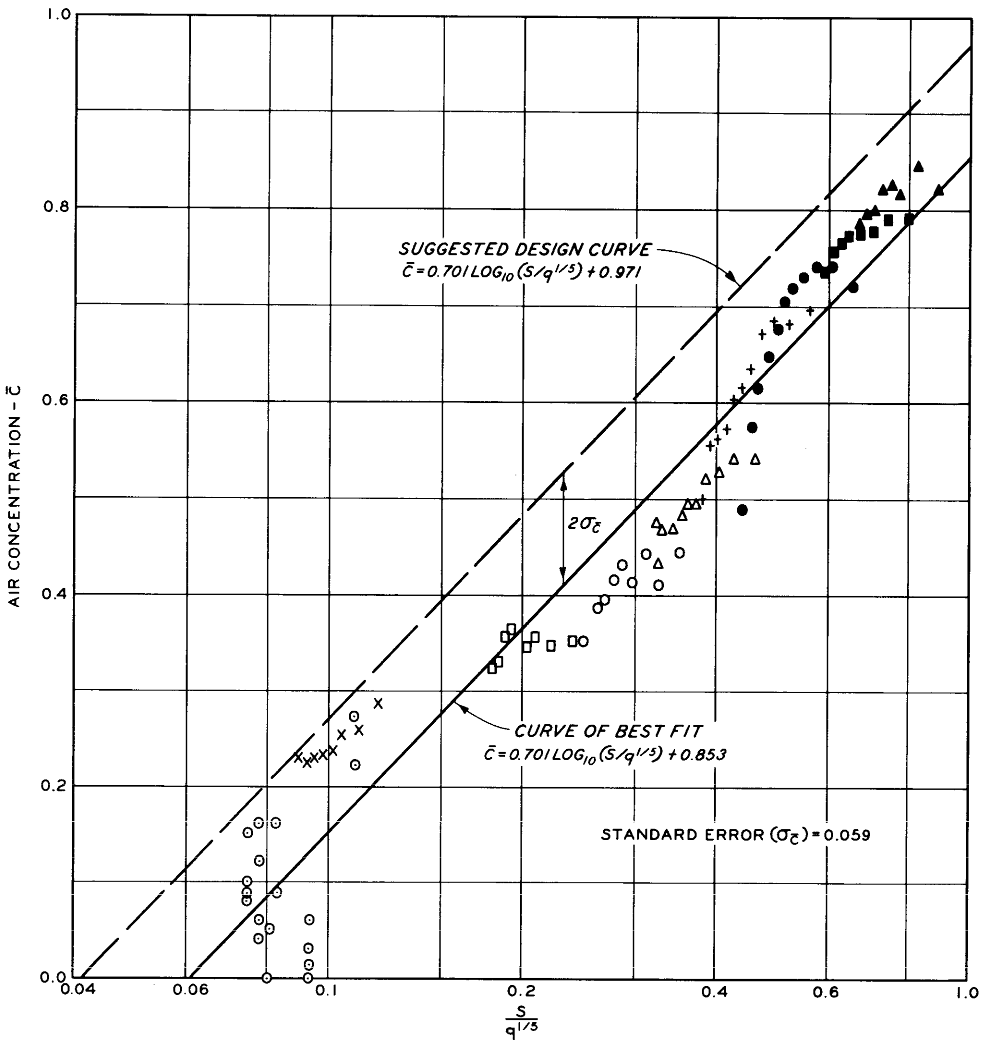
5. Suggested Criteria. The curve of best fit in HDC 050-3 was determined by the least squares method using both the Minnesota and Kittitas data. The suggested design curve is believed to be a conservative basis for design. The results are applicable to flow at terminal velocity in chute spillways having width-depth ratios greater than five.

6. References.

(1) ASCE Committee on Hydromechanics, "Aerated flow in open channels." Progress Report, Task Committee on Air Entrainment in Open Channels, Proceedings, American Society of Civil Engineers, vol 87, part 1 (Journal, Hydraulics Division, No. HY3) (May 1961).

(2) Hall, L. S., "Open channel flow at high velocities." Transactions, American Society of Civil Engineers, vol 108 (1943), pp 1394-1434 and 1494-1513.

(3) Straub, L. G., and Anderson, A. G., "Self-aerated flow in open channels." Transactions, American Society of Civil Engineers, vol 125 (1960), pp 456-486.



NOTE: $\bar{C}$ = RATIO OF AIR VOLUME TO AIR-PLUS-WATER VOLUME
q = DISCHARGE PER UNIT WIDTH, CFS
S = SINE OF ANGLE OF CHUTE INCLINATION

LEGEND

MINNESOTA DATA
- × S = 0.13
- □ S = 0.26
- ○ S = 0.38
- △ S = 0.50
- + S = 0.61
- ● S = 0.71
- ■ S = 0.87
- ▲ S = 0.97

KITTITAS DATA
- ⊙ S = 0.18

AIR ENTRAINMENT
WIDE CHUTE FLOW
CONCENTRATION ($\bar{C}$) VS $S/q^{1/5}$

HYDRAULIC DESIGN CHART 050-3

HYDRAULIC DESIGN CRITERIA

SHEETS 060-1 TO 060-1/5

GATE VIBRATION

1. Purpose. One of the problems in the design of reservoir outlet structures is the determination of whether any disturbing frequencies are inherent in the hydraulic system that may equal or approach the natural frequency of the gate and cause resonance with resulting violent gate vibrations. Although a gate leaf may vibrate in any of several freedoms of motion including flexure, the vertical vibration of a gate on an elastic suspension is usually of most importance. Hydraulic Design Charts 060-1 to 060-1/5 are aids for estimating the vibration characteristics of elastically suspended gates.

2. Resonance. When the forcing frequency is exactly equal to the natural frequency a condition of "dead" resonance exists. The displacement amplitude for the vibrating system increases very rapidly for this condition of resonance and may result in rupture. The amplitude can also be increased rapidly if there is only a small difference between the forcing and natural frequencies. The transmissibility ratio, or the magnification factor, is defined by the equation:

$$T.R. = \frac{1}{1 - (f_f/f_n)^2}$$

where f_f/f_n is the ratio of the forcing frequency to natural frequency. A plot and coordinates of this function are given on Hydraulic Design Chart 060-1. Although the transmissibility ratio is negative for frequency ratios greater than one, the positive image of this part of the curve is often utilized for simplicity in plotting. The part of the curve between transmissibility ratios of unity and zero is sometimes called the isolation range with the percentage of isolation as designated. It is desirable to produce a design with a high percentage of isolation.

3. Forcing Frequencies. Two possible sources of disturbing frequencies are the vortex trail shed from the bottom edge of a partly opened gate and the pressure waves that travel upstream to the reservoir and are reflected back to the gate. The frequency of the vortex trail shed from a flat plate can be defined by the dimensionless Strouhal number, S_t, as follows:

$$S_t = \frac{L_p f_f}{V}$$

where L_p is the plate width, f_f is the vortex trail shedding frequency, and V is the velocity of the fluid. The Strouhal number for a flat plate is approximately 1/7. The forcing frequency of a vortex trail shed from a gate may be estimated as:

$$f_f = \frac{\sqrt{2gH_e}}{7(2Y)}$$

where H_e is the energy head at the bottom of the gate, and Y is the projection of the gate into the conduit or half of the plate width L_p. Hydraulic Design Chart 060-1/1 can be used to estimate the forcing frequency for various combinations of energy head and gate projection. Unpublished observations of hydraulic models of gates have indicated that the vortex trail will spring from the upstream edge of a flat-bottom gate causing pressure pulsations on the bottom of the gate. The vortex trail springs from the downstream edge of a standard 45-degree gate lip, eliminating bottom pulsations.

4. The frequency of a reflected positive pressure wave may be determined from the equation:

$$f_f = \frac{C}{4L}$$

where C is the velocity of the pressure wave and L is the length of the conduit upstream from the gate. Hydraulic Design Chart 060-1/2 is a graphical solution of this equation. The pressure wave velocity is dependent upon the dimensions and elastic characteristics of the pipe or of the lining and surrounding rock of a tunnel. Data are given by Parmakian* for various combinations of these variables. Chart 060-1/2 gives frequencies for pressure wave velocities ranging from 4700 fps for a relatively inelastic conduit to 3000 fps for a relatively elastic pipe.

5. Natural Frequency. The natural frequency of free vertical oscillation of a cable-suspended gate can be expressed by the equation:

$$f_n = \frac{1}{2\pi} \sqrt{\frac{gE}{12\,\ell\,\sigma}}$$

where E is the modulus of elasticity of the cable, ℓ is the length of the supporting cable, and σ is the unit stress in the cable. The natural frequencies for various support lengths and typical allowable unit stresses can be estimated from Hydraulic Design Chart 060-1/3.

6. Examples of Application. Hydraulic Design Charts 060-1/4 and 1/5 are sample computations illustrating application of Charts 060-1 to 1/3 to the gate vibration problem. Transmissibility ratios less than 1.0 are desirable. However, ratios slightly greater than 1.0 may be satisfactory if the vibration forces are damped.

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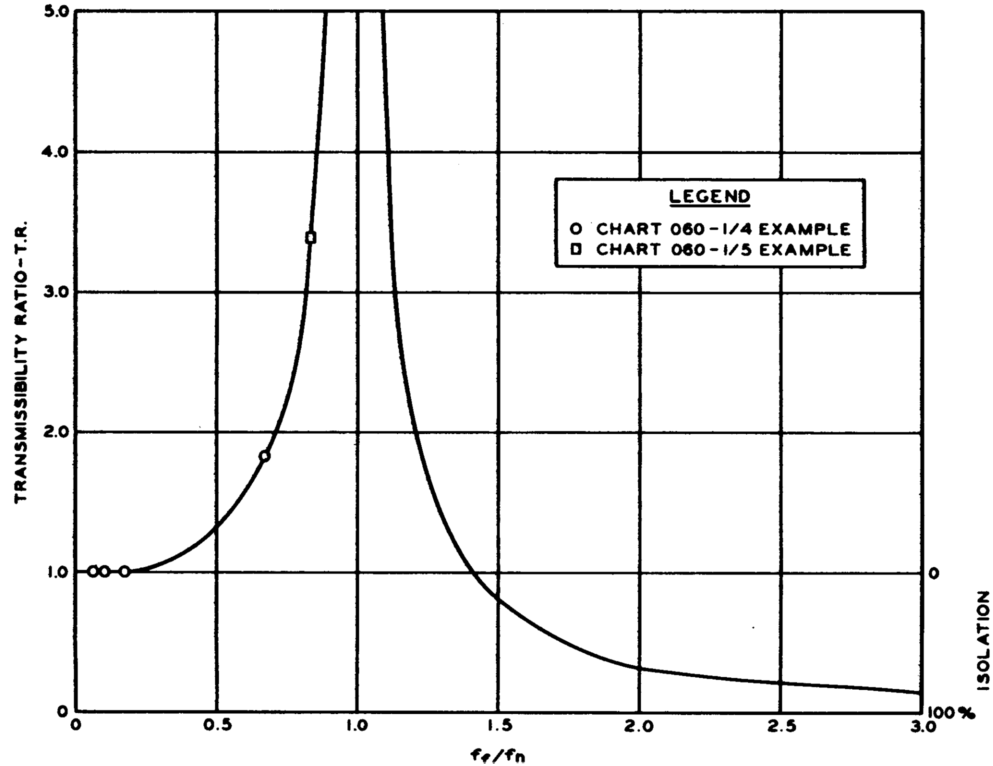
* John Parmakian, Waterhammer Analysis, 1st ed. (New York, Prentice-Hall, Inc., 1955), Chap. 3.



f_f/f_n	T.R.	f_f/f_n	T.R.	f_f/f_n	T.R.
0.00	1.000	0.85	3.604	1.50	0.800
0.10	1.010	0.90	5.263	1.60	0.641
0.20	1.042	0.95	10.256	1.70	0.529
0.30	1.099	1.05	9.756	1.80	0.446
0.40	1.191	1.10	4.762	1.90	0.383
0.50	1.333	1.15	3.101	2.00	0.333
0.60	1.563	1.20	2.273	2.20	0.260
0.65	1.732	1.25	1.778	2.40	0.210
0.70	1.961	1.30	1.449	2.60	0.174
0.75	2.286	1.35	1.216	2.80	0.146
0.80	2.778	1.40	1.042	3.00	0.125

BASIC EQUATION

$$T.R. = \frac{1}{1-(f_f/f_n)^2}$$

WHERE:

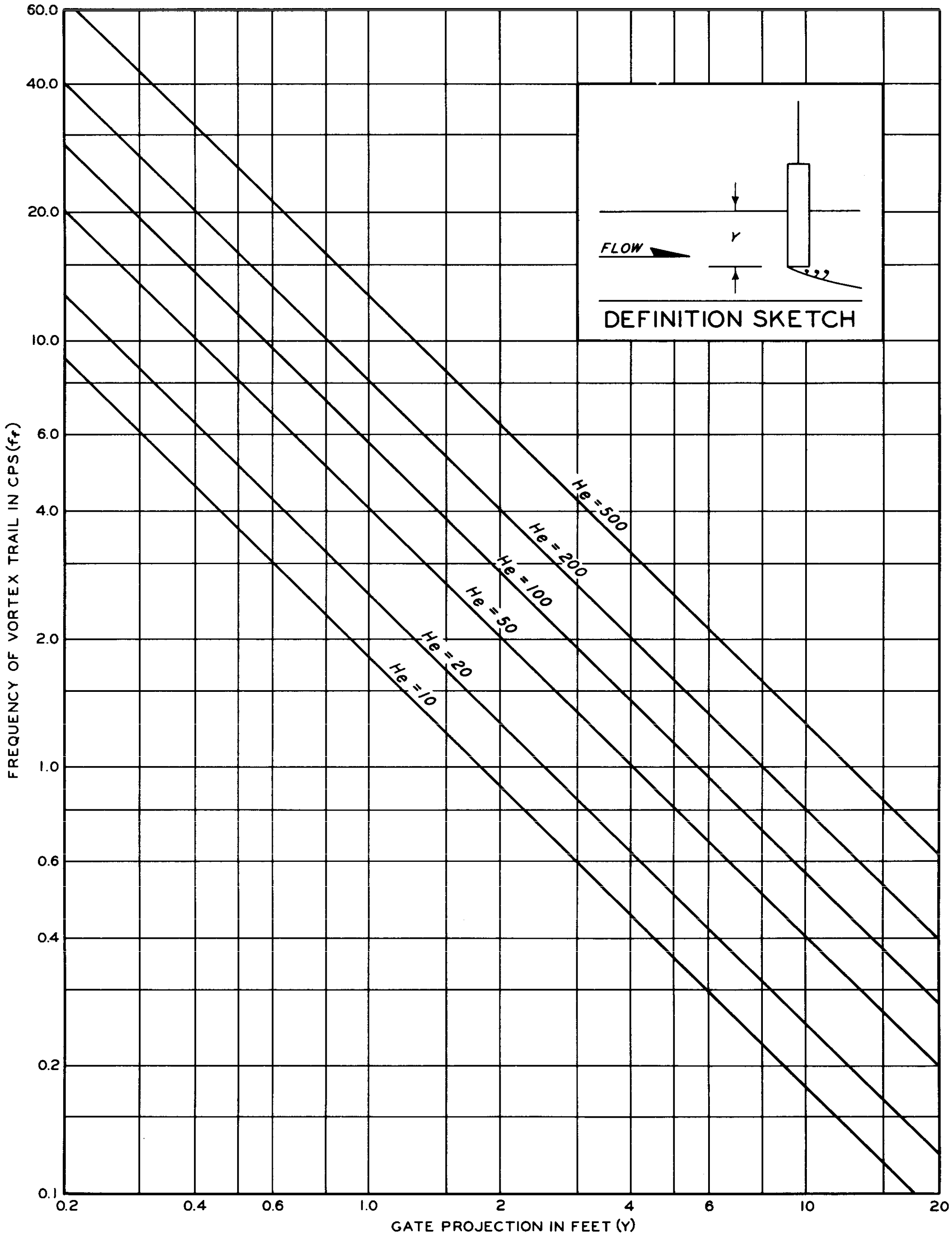
T.R. = TRANSMISSIBILITY RATIO
f_f = FORCING FREQUENCY
f_n = NATURAL FREQUENCY

GATE VIBRATION

RESONANCE DIAGRAM

HYDRAULIC DESIGN CHART 060-1

WES 6-57



BASIC EQUATION $f_f = \dfrac{\sqrt{2gH_e}}{7(2Y)}$

WHERE:

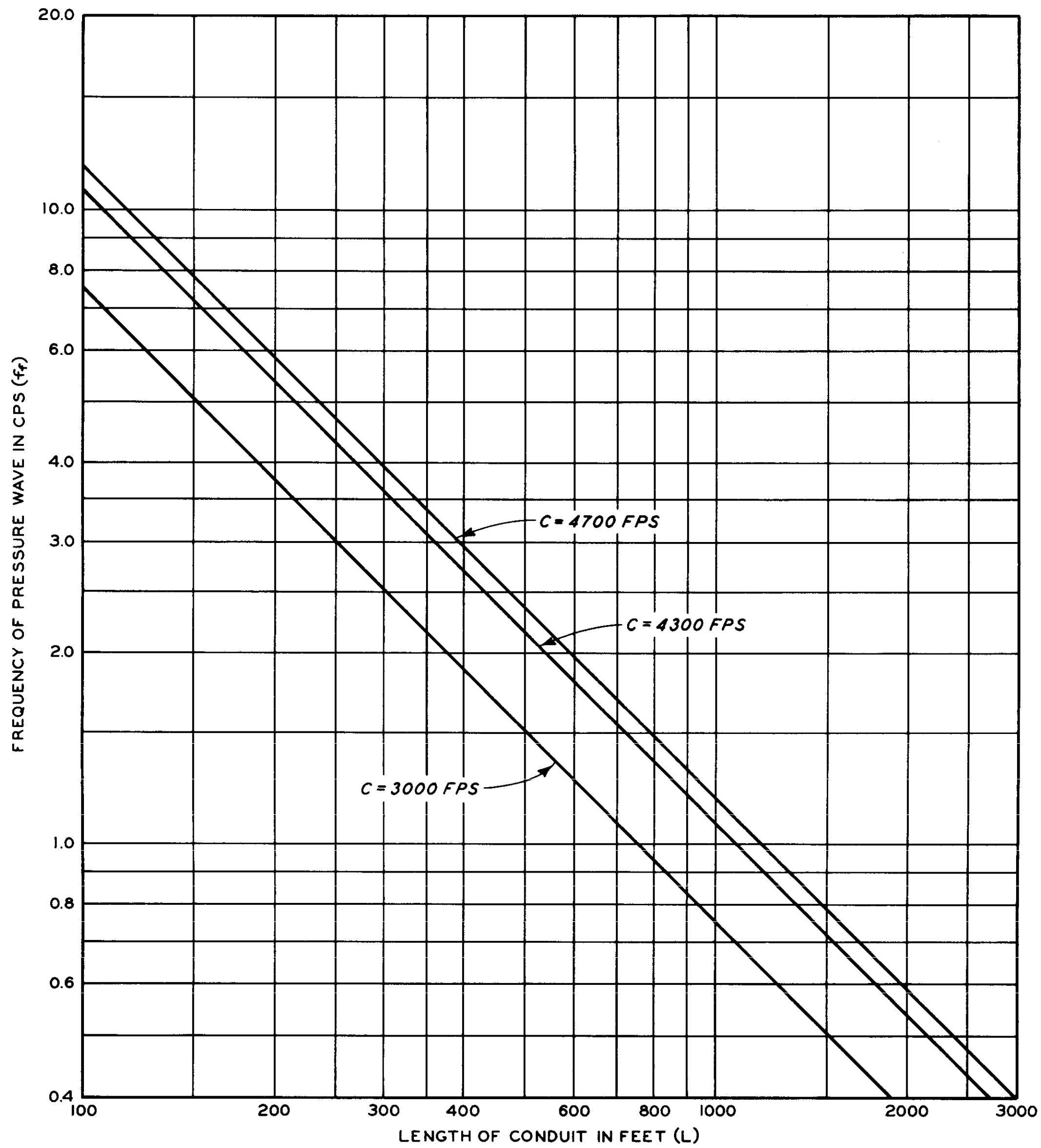
- f_f = FORCING FREQUENCY IN CPS
- H_e = ENERGY HEAD IN FT TO BOTTOM OF GATE
- 1/7 = STROUHAL NO. FOR FLAT PLATE
- Y = GATE PROJECTION IN FT

GATE VIBRATION

VORTEX TRAIL - FORCING FREQUENCY

HYDRAULIC DESIGN CHART 060-1/1

WES 6-57



BASIC EQUATION $f_f = \frac{C}{4L}$

WHERE:

f_f = FREQUENCY OF PRESSURE WAVE IN CPS

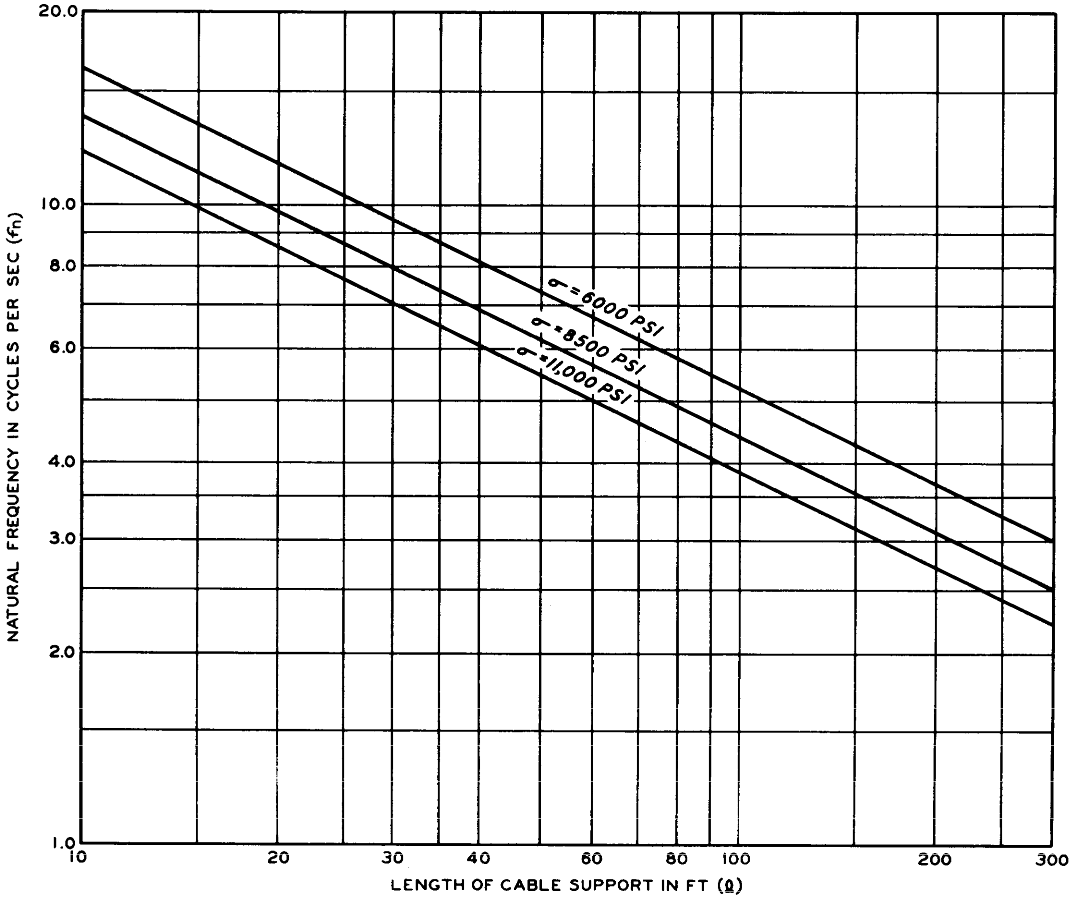
C = VELOCITY OF PRESSURE WAVE - FPS

L = LENGTH OF CONDUIT UP-STREAM FROM GATE IN FT

GATE VIBRATION

FORCING FREQUENCY OF REFLECTED PRESSURE WAVE

HYDRAULIC DESIGN CHART 060-1/2



BASIC EQUATION

$$f_n = \frac{1}{2\pi}\sqrt{\frac{gE}{12\ell\sigma}}$$

WHERE:

f_n = NATURAL FREQUENCY OF SUSPENDED SYSTEM IN CYCLES PER SEC

g = ACCELERATION OF GRAVITY = 386 IN./SEC^2

E = MODULUS OF ELASTICITY OF CABLE = 20×10^6 PSI

ℓ = LENGTH OF CABLE SUPPORT IN FT

σ = UNIT STRESS IN CABLE STEEL IN PSI

GATE VIBRATION

NATURAL FREQUENCY OF CABLE - SUSPENDED GATE

HYDRAULIC DESIGN CHART 060-1/3

WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Gate Vibration

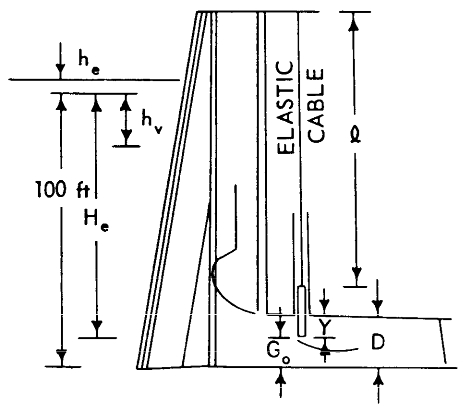
COMPUTATION Vibration From Vortex Trail

COMPUTED BY RGC DATE 4/16/57 CHECKED BY RGC DATE 4/24/57

GIVEN:

Gate - flat bottom
- Height (D) = 23 ft
- Projection (Y) into conduit
  - = height minus gate opening
  - = $D - G_o$
- Length of cable (ℓ) = 130 ft
- Allowable unit cable stress (σ)
  - = 8500 psi

Total head at gate sill = 100 ft



DETERMINE:

Natural frequency (f_n) for gate:
- Length of cable (ℓ) = 130 ft
- Unit cable stress (σ) = 8500 psi
- From Chart 060-1/3 natural frequency
  - (f_n) = 3.8 cps

Vortex trail frequency and resonance characteristics (f_f/f_n):
- Energy head (H_e) to bottom of gate = 100 ft − G_o.

Gate Opening G_o	Gate Projection $Y = D - G_o$	H_e $100 - G_o$	Vortex Trail Frequency f_f (Chart 060 − 1/1)	Resonance Characteristics (f_f/f_n)
3	20	97	0.28	0.07
9	14	91	0.39	0.10
15	8	85	0.66	0.17
21	2	79	2.54	0.67

Plot f_f/f_n on Chart 060-1:

All points plot above zero isolation line. Gate subject to vibration at all openings. Change design to 45 degree gate lip.

GATE VIBRATION

GATE BOTTOM VORTEX TRAIL SAMPLE COMPUTATION

WATERWAYS EXPERIMENT STATION

COMPUTATION SHEET

JOB CW 804 PROJECT John Doe Dam SUBJECT Gate Vibration

COMPUTATION Vibration From Reflected Pressure Wave

COMPUTED BY RGC DATE 4/22/57 CHECKED BY RGC DATE 4/25/57

GIVEN:

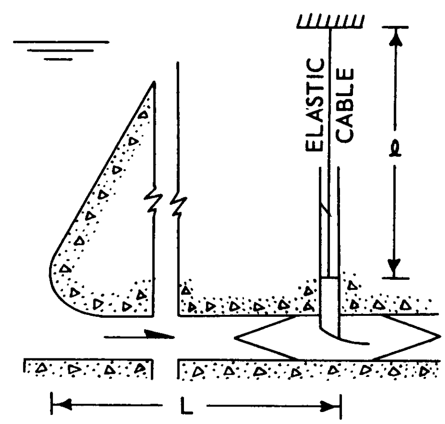
Conduit:

Length upstream from gate (L) = 400 ft

Gate:

Length of supporting cable (ℓ) = 195 ft

Assume unit stress in supporting cable (σ) = 8500 psi



DETERMINE:

Natural frequency (f_n) for gate:

Length of supporting cable (ℓ) = 195 ft

Unit stress in supporting cable (σ) = 8500 psi

From Hydraulic Design Chart 060-1/3, natural frequency (f_n) = 3.2 cps

Vibration from reflected pressure wave:

Length of conduit upstream from gate (L) = 400 ft

Velocity of pressure wave (C) = 4300 fps (assumed for concrete conduit through rock)

From Hydraulic Design Chart 060-1/2, forcing frequency (f_f) = 2.7 cps

Resonance characteristics:

$f_f/f_n = 2.7/3.2 = 0.84$

Plot f_f/f_n on Hydraulic Design Chart 060-1

Transmissibility ratio (T.R.) = 3.4

Isolation < 0

Gate subject to vibration from reflected pressure wave if undamped. Damping forces not evaluated.

GATE VIBRATION

REFLECTED PRESSURE WAVE

SAMPLE COMPUTATION

HYDRAULIC DESIGN CHART 060-1/5

WES 6-57

HYDRAULIC DESIGN CRITERIA

SHEET 060-2

FORCED VIBRATIONS

CONSTANT FRICTION DAMPING

1. A procedure for estimating the vibration characteristics of free, elastically suspended gates is presented in HDC's 060-1 to 060-1/5. Tests at Fort Randall Dam[2] indicated that at large gate openings the flood-control tunnel gate rollers are momentarily forced away from the gate guides by pressure pulsations on the downstream face of the gate. Vertical gate vibrations of about 4 cycles per sec were observed during this interval. These vibrations were damped when the gate rollers returned to the guides. It was determined that the damping was of a Coulomb or constant friction damping character.

2. HDC 060-2 presents curves showing the effects of constant friction damping on forced vibrations. The equation and curves on the chart for the magnification factor were developed by Den Hartog[1] in 1931. The equation is only of value in the determination of the magnification factor above the dashed line on the chart. Den Hartog also successfully evaluated single points below the dashed line and constructed the curves representing high force ratios. More recently (1960) an analysis of the damping forces affecting the Fort Randall gates has been made.[3]

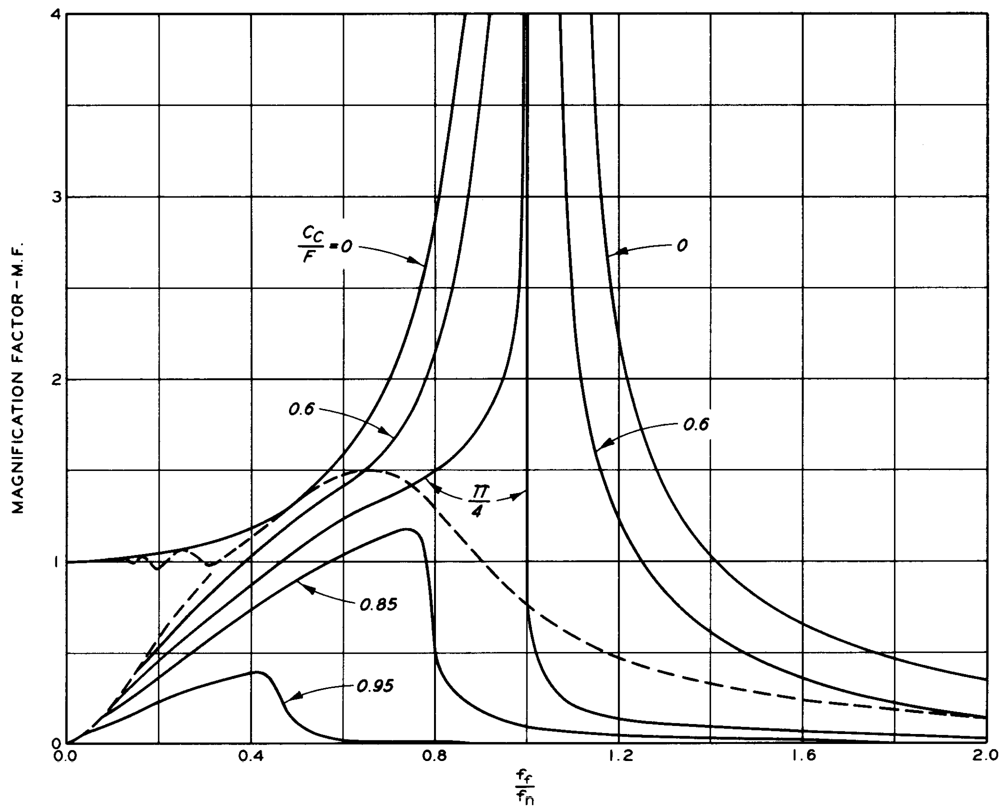
3. Field measurements to determine the causes, magnitudes, and frequencies of hydraulic disturbances causing gate vibration, as well as measurements of the resisting friction forces, are necessary for detail evaluation of the vibration characteristics of these hydraulic structures. HDC 060-2 is included as a supplement to HDC 060-1 to illustrate the effects of constant friction damping in the gate vibration problem.

4. References.

(1) Den Hartog, J. P., "Forced vibration with combined Coulomb and viscous friction." Transactions, American Society of Mechanical Engineers, Paper APM 53-9, presented at National Applied Mechanics Meeting, Purdue University (June 1931).

(2) U. S. Army Engineer Waterways Experiment Station, CE, Vibration and Pressure-Cell Tests, Flood-Control Intake Gates, Fort Randall Dam, Missouri River, South Dakota. Technical Report No. 2-435, Vicksburg, Miss., June 1956.

(3) _________, Vibration Problems in Hydraulic Structures, by F. B. Campbell. Miscellaneous Paper No. 2-414, Vicksburg, Miss., December 1960. Also in Proceedings, American Society of Civil Engineers, vol 87 (Journal, Hydraulics Division, No. HY2) (March 1961).



EQUATIONS

$$M.F. = \sqrt{A^2 - \frac{C_C^{\,2}}{F^2} B^2}$$

$$A = \frac{1}{1 - \frac{f_f^{\,2}}{f_n^{\,2}}}$$

$$B = \frac{f_n}{f_f} \text{TAN} \frac{\pi f_n}{2 f_f}$$

WHERE:

M.F. = MAGNIFICATION FACTOR

C_C = CONSTANT FRICTION FORCE, LB

F = EXCITING FORCE, LB

f_f = FORCING FREQUENCY, CPS

f_n = NATURAL FREQUENCY, CPS

FORCED VIBRATIONS

CONSTANT FRICTION DAMPING

HYDRAULIC DESIGN CHART 060 - 2

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